PROCEEDINGS

American Society of Civil Engineers

JUNE
1940

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

VOLUME 66
NO. 6

TWO PARTS
PART 1

PROCEEDINGS

OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

SYDNEY WILMOT
Manager of Publications

HAROLD T. LARSEN
Editor of Proceedings

COMMITTEE ON PUBLICATIONS

THOMAS R. AGG, *Chairman*

LOUIS E. AYRES
CHARLES T. LEEDS
HAROLD M. LEWIS
LAZARUS WHITE

Editorial Office, 33 West 39th Street, New York, N. Y.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 66 | JUNE, 1940 | NO. 6, PART 1

TECHNICAL PAPERS

AND

DISCUSSIONS

Published monthly, except July and August, at Prince and Lemon Streets, Lancaster, Pa., by the American Society of Civil Engineers. Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, September 23, 1937, at the Post Office at Lancaster, Pa., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy.

CURRENT PAPERS AND DISCUSSIONS

		Discussion closes
Stress Distribution Around a Tunnel. *Raymond D. Mindlin*	Apr., 1939	
Discussion	Oct., 1939, Feb., June, 1940	Closed
Reconstruction of the Walpole-Bellows Falls Arch Bridge. *H. E. Langley and Edward J. Ducey*	Apr., 1939	
Discussion	Sept., Oct., 1939, Jan., 1940	Closed*
Flash-Board Pins. *Chilton A. Wright and Clifford A. Betts*	May, 1939	
Discussion	Nov., Dec., 1939, Jan., Apr., June, 1940	Closed
Tension Tests of Large Riveted Joints. *Raymond E. Davis, Glenn B. Woodruff, and Harmer E. Davis*	May, 1939	
Discussion	Sept., Oct., 1939, Apr., May, June, 1940	Closed
Combining Geodetic Survey Methods with Cadastral Surveys. *Carl M. Berry*	Sept., 1939	
Discussion	Dec., 1939, Jan., Feb., June, 1940	Closed
An Improved Method for Adjusting Level and Traverse Surveys. *Clarence Norris and Julius L. Speert*	Oct., 1939	
Discussion	Jan., Feb., Mar., June, 1940	Closed
The Unit Hydrograph Principle Applied to Small Water-Sheds. *E. F. Brater*	Sept., 1939	
Discussion	Jan., Feb., Apr., June, 1940	Closed
Development of the Colorado River in the Upper Basin. *Thomas C. Adams*	Sept., 1939	
Discussion	May, June, 1940	Closed
Field Tests of a Shale Foundation. *August E. Niederhoff*	Sept., 1939	
Discussion	Jan., Feb., June, 1940	Closed
General Wedge Theory of Earth Pressure. *Karl Terzaghi*	Oct., 1939	
Discussion	Jan., Feb., Apr., June, 1940	Closed*
Functional Design of Flood Control Reservoirs. *C. J. Posey and Fu-Te I*	Oct., 1939	
Discussion	Dec., 1939, Mar., Apr., May, 1940	Closed*
Sewage Disposal Project of Buffalo, New York. *Samuel A. Greeley*	Oct., 1939	
Discussion	Nov., 1939, Apr., May, 1940	Closed*
Relation of the Statistical Theory of Turbulence to Hydraulics. *A. A. Kalinske*	Oct., 1939	
Discussion	Jan., Feb., Mar., Apr., May, June, 1940	Closed*
Effective Moment of Inertia of a Riveted Plate Girder. *Scott B. Lilly and Samuel T. Carpenter*	Oct., 1939	
Discussion	Dec., 1939, Jan., Feb., Mar., May, June, 1940	Closed
Problems and Trends in Activated Sludge Practice. *Robert T. Regester*	Nov., 1939	
Discussion	Mar., Apr., June, 1940	Closed*
The Rôle of the Engineer in Air Sanitation: A Symposium	Nov., 1939	
Discussion	Feb., June, 1940	Closed*
Water Supply on Upper Salt River, Arizona. *John Girand*	Dec., 1939	
Discussion	Mar., Apr., June, 1940	Closed*
Bridge and Tunnel Approaches. *John F. Curtin*	Nov., 1939	
Discussion	Mar., Apr., 1940	June, 1940
Trend in Hydraulic Turbine Practice: A Symposium	Nov., 1939	
Discussion	Jan., Mar., Apr., May, 1940	June, 1940
Effects of Rifling on Four-Inch Pipe Transporting Solids. *G. W. Howard*	Nov., 1939	
Discussion	Mar., Apr., May, 1940	June, 1940
Transient Flood Peaks. *Henry B. Lynch*	Nov., 1939	
Discussion	Jan., Feb., Mar., Apr., May, June, 1940	June, 1940
Channelization of Motor Traffic. *Guy Kelcey*	Dec., 1939	
Discussion	Mar., Apr., May, June, 1940	June, 1940
Analysis of Legal Concepts of Subflow and Percolating Waters. *C. F. Tolman and Amy C. Stipp*	Dec., 1939	
Discussion	Feb., Apr., May, June, 1940	June, 1940
Miniature System of First-Order Alinement and Triangulation Control. *Floyd W. Hough*	Dec., 1939	
Discussion	Apr., June, 1940	June, 1940
Pressure-Momentum Theory Applied to the Broad-Crested Weir. *H. A. Doeringsfeld and C. L. Barker*	Dec., 1939	
Discussion	Mar., Apr., June, 1940	June, 1940
Norris Dam Construction Cableways. *R. T. Colburn and L. A. Schmidt, Jr.*	Dec., 1939	
Discussion	Mar., Apr., June, 1940	June, 1940
Standards of Professional Relations and Conduct. *Daniel W. Mead*	Jan., 1940	
Discussion	Mar., Apr., June, 1940	June, 1940
Theory of Elastic Stability Applied to Structural Design. *Leon S. Moisseiff and Frederick Lienhard*	Jan., 1940	June, 1940
Chicago River Control Works. *H. P. Ramey*	Jan., 1940	
Discussion	June, 1940	June, 1940
Axioms in Roadway Soil Mechanics. *Henry C. Porter*	Feb., 1940	June, 1940
Measuring the Potential Traffic of a Proposed Vehicular Crossing. *N. Cherniack*	Feb., 1940	
Discussion	June, 1940	June, 1940
Sealing the Lagoon Lining at Treasure Island with Salt. *Charles H. Lee*	Feb., 1940	
Discussion	June, 1940	June, 1940
Progress Report of Committee of the Hydraulics Division on Flood Control	Feb., 1940	
Discussion	May, June, 1940	June, 1940
Foundation Experiences, Tennessee Valley Authority: A Symposium	Mar., 1940	
Discussion	May, June, 1940	July, 1940
Design of Hinges and Articulations in Reinforced Concrete Structures. *George C. Ernst*	Apr., 1940	Aug., 1940
Permissible Composition and Concentration of Irrigation Water. *W. P. Kelley*	Apr., 1940	Aug., 1940
Progress Report of the Committee on Flood-Protection Data	Apr., 1940	
Discussion	June, 1940	Aug., 1940
Progress Report of the Committee of the Structural Division on Oxygen Cutting (Flame Cutting) of Structural Steel	Apr., 1940	Aug., 1940
Masonry Dams: A Symposium	May, 1940	Sept., 1940

NOTE.—The closing dates herein published are final except when names of prospective discussers are registered for special extension of time.

* Publication of closing discussion pending.

CONTENTS FOR JUNE, 1940

PAPERS

CONTENTS FOR JUNE, 1940 (*Continued*)

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

MODEL TESTS, BRIDGE PIER SUPPORTED ON LONG STEEL PILES

By Thomas F. Comber, Jr.,[1] M. Am. Soc. C. E., and John M. Coan, Jr.,[2] Jun. Am. Soc. C. E.

Synopsis

Many theories[3] on the stress analysis of groups of long, unsupported piles have been advanced without adequate substantiation by test data. This paper gives the results of tests on a model of a proposed deep-water bridge pier supported on long, tubular steel piles. The model conformed to the prototype except for the neglect of the restraining action of the foundation material. Pile stresses and pier deflections were measured in the model from which corresponding values may be determined in the prototype.

These test data are submitted in the hope that they may provide a check for one of the previously advanced theoretical analyses, or may lead to a new treatment, or, in any case, may add to the knowledge of the structural behavior of such pile groups.

Introduction

In many locations where bedrock is heavily overlaid with alluvial deposits and deep water, important river crossings are prohibited by the high cost of pier construction resulting from the use of deep caissons or cofferdams. This economic disadvantage might be overcome by the use of piers consisting of a concrete pier cap, supported on groups of long piles, with the elevation of the base of the cap only slightly below mean low water. In such a pier, construction difficulties would be simplified greatly and the quantities of materials much reduced, thus effecting large economies.

The pier which served as a prototype for the test model is shown in Fig. 1. This pier was selected for these tests because of its contemplated use in an actual bridge design. Strains and deflections were measured in the model to determine the probable stresses and deflections in the prototype.

Note—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by October 15, 1940.

[1] Associate Prof. of Civ. Eng., The Johns Hopkins Univ., Baltimore, Md.

[2] Stress Analyst, The Glenn L. Martin Company, Middle River, Md.

[3] See various theories in "Design of Pile Foundations," by C. P. Vetter, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 758.

The Model

The model was made in accordance with the principles of similitude.[4] In Fig. 2(*c*), the numerals in parentheses indicate the unrestrained lengths of the piles from cap to base. The companion values, not in parentheses, are the

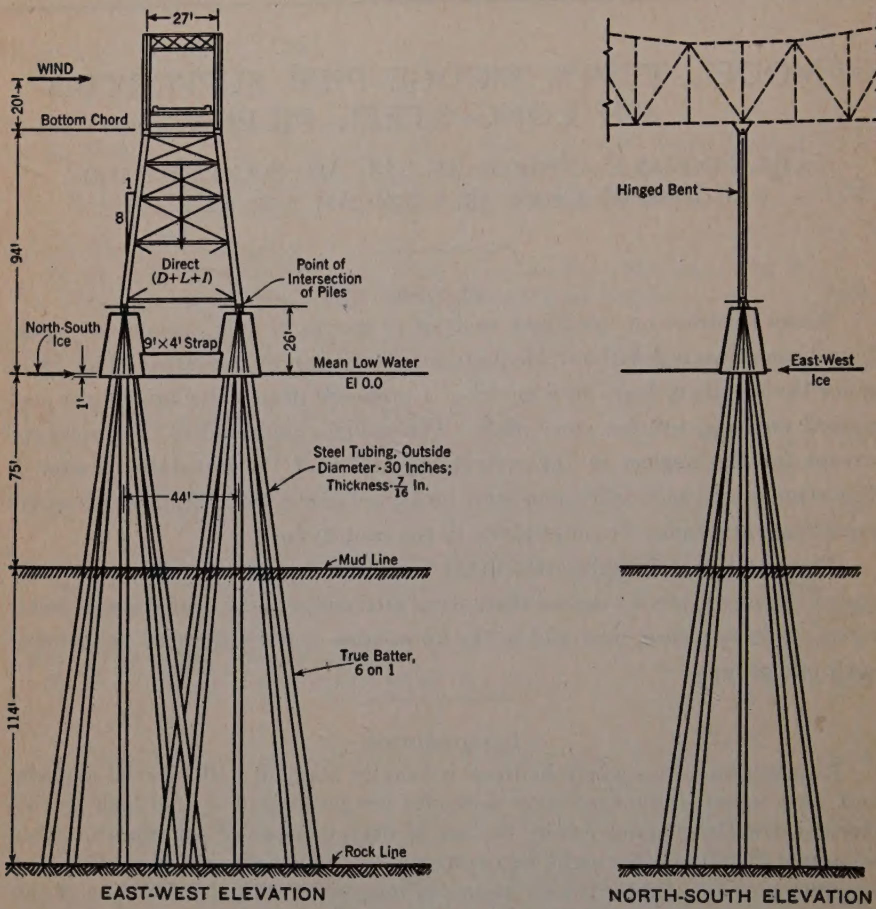


Fig. 1.—Views of Prototype

lengths of the piles from the base to their theoretical point of intersection. The approximate length-reduction factor was set by considerations of economy, convenience, etc., to be about forty, and it was established definitely by the available commercial sizes of steel tubing. The contemplated pier design called for piles of steel tubing with an outside diameter of 30 in., a shell thickness of $\frac{7}{16}$ in., and an approximate length of 200 ft. The most convenient tubing for use in the model had an outside diameter of 0.7565 in. and a thickness of 0.030 in. The inside and outside diameters of the ten piles used were mea-

[4] "Similitude Requirements in Model Design," by Roy W. Carlson, Assoc. M. Am. Soc. C. E., *Engineering News-Record*, August 23, 1934.

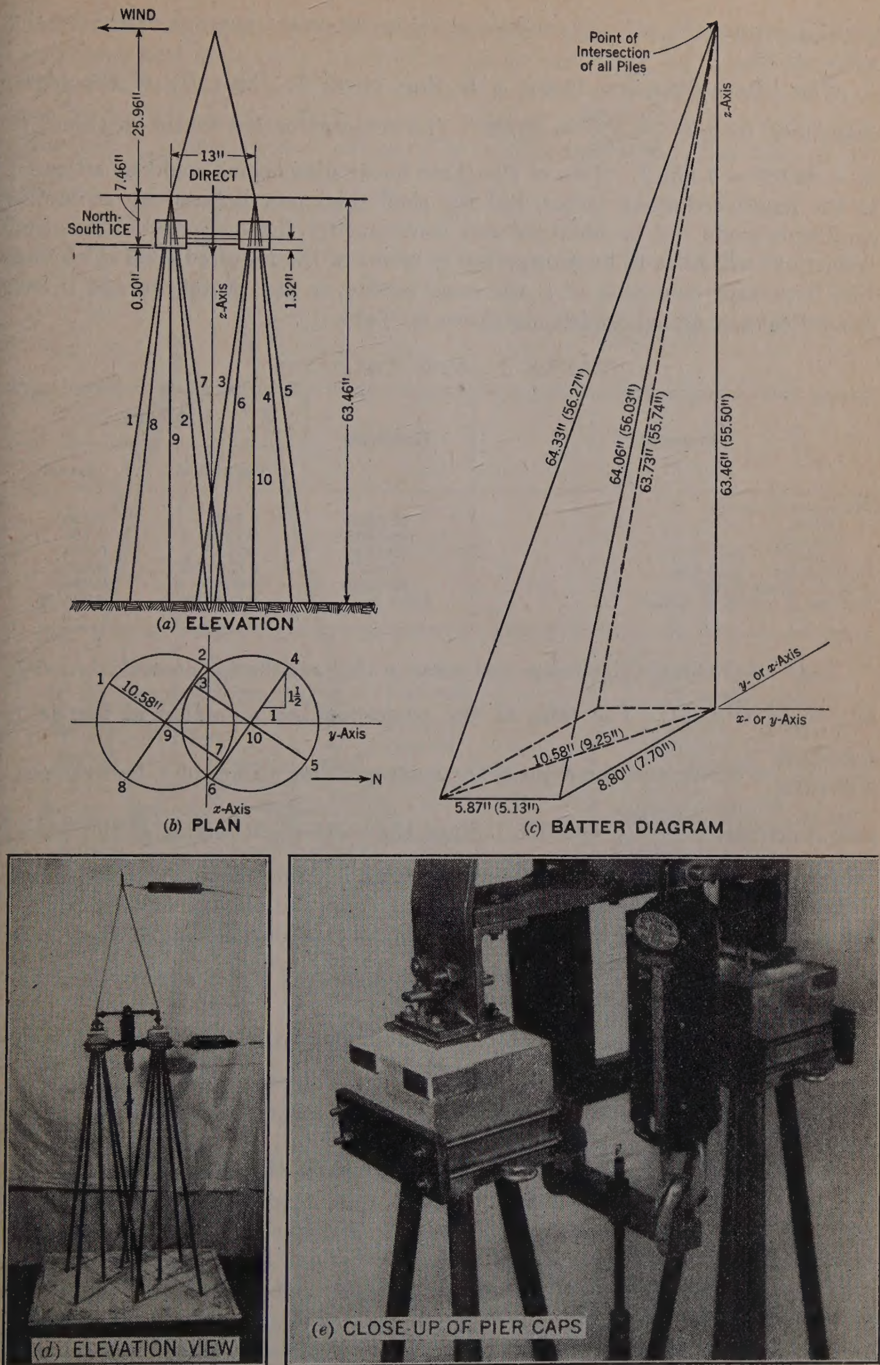


Fig. 2.—Views of Model

sured accurately by calipers, and these values represent average values for the entire group.

The length-reduction factor λ is thus equal to the ratio of the mean diameters, or $\lambda = \frac{29.5625}{0.7265} = 40.69$. The force-reduction factor is therefore $\lambda^2 = 40.69^2 = 1{,}655.7$. Two of the three linear dimensions could be adjusted to the length-reduction factor, but the shell thickness dictated by geometric similitude could not be obtained and consequently the measured strains and deflections will have to be interpreted in terms of the required shell thickness. The important constants of a pile cross section in the prototype and in the theoretical and actual models are shown in Table 1.

TABLE 1.—PILE CONSTANTS

Properties	Prototype	Model	
		Theoretical	Actual
Diameters, in inches			
Mean	29.5625	0.7265	0.7265
Outside	30.0000	0.7373	0.7565
Inside	29.1250	0.7157	0.6965
Wall thickness, in inches	0.4375	0.01075	0.0300
Area, in square inches	40.6322	0.02454	0.06847
Moment of inertia, in inches[4]	4,439.74	0.001619	0.004525

The ratio of shell thicknesses and areas in the actual and theoretical models is $\frac{0.0300}{0.01075} = 2.791$. The ratio of the corresponding moments of inertia is $\frac{0.004525}{0.001619} = 2.795$, and the ratio of the corresponding values of c, half the outside diameter, is $\frac{0.7565}{0.7373} = 1.026$. Since the section properties of the actual model are greater than required by similitude, the measured strains and deflections will be less than in the theoretical model and must be modified by the foregoing ratios. Since the stresses due to axial load and bending moment vary as the values of A and $\frac{I}{c}$, respectively, the measured stresses must be multiplied by 2.791, and $\frac{2.795}{1.026} = 2.724$, to obtain the stresses in the theoretical model and, hence, the prototype. The deflection at any point in the prototype will be $2.795\,\lambda = 113.73$ times the observed value at a corresponding point in the model.

No attempt was made in these tests to correct for the inadequate density of the piles because, before the testing gages were applied, the model had adjusted itself to its own weight and that of the permanently attached apparatus (see Figs. 2(*d*) and 2(*e*)). The initial deformations and deflections thus induced were not measured, the gage readings indicating only the effects of the externally applied loads.

The concrete caps were assumed to be inelastic, no account being taken of their internal deformations. The action of the entire structure will depend,

however, on the elastic properties of the strut connecting the caps, and its dimensions were adjusted as required by similitude. Had the strut been made of concrete in accordance with the prototype plans, its dimensions would have been $2\frac{5}{8}$ in. by $1\frac{3}{16}$ in. Such a small member would be impracticable and, accordingly, a steel strut of comparable rigidity was used. The ratio of the

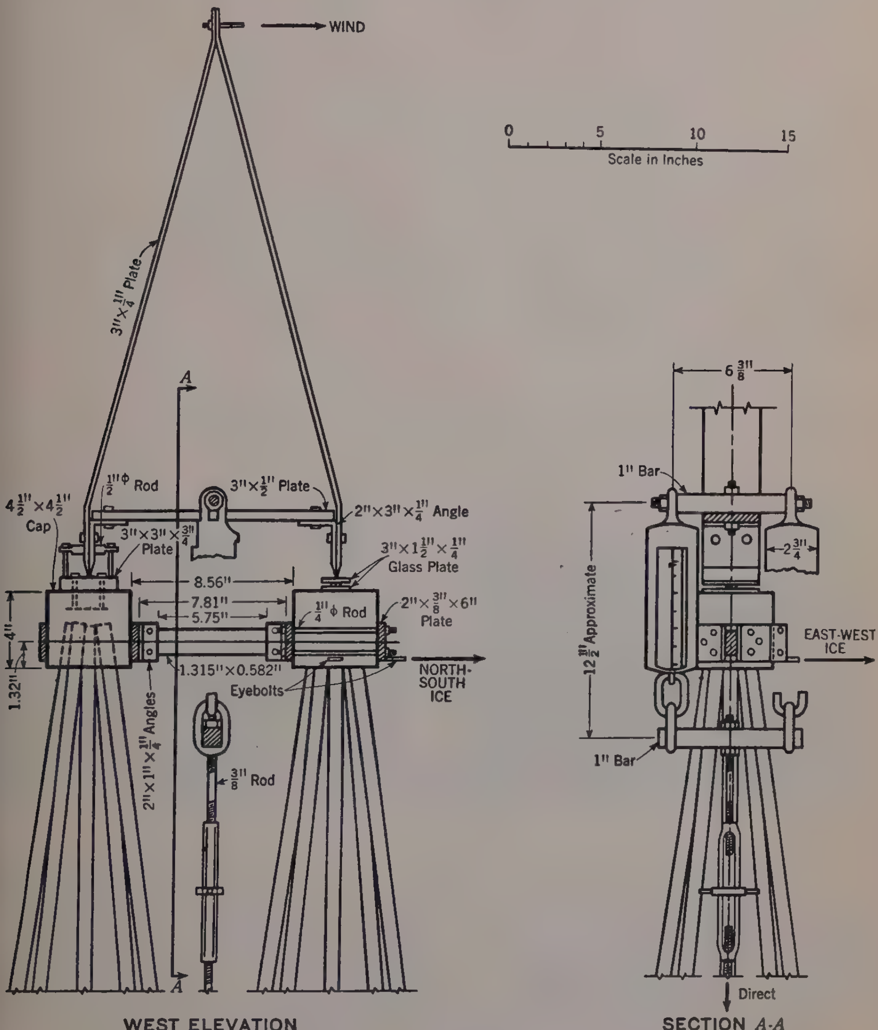


Fig. 3.—Pier Cap and Associated Apparatus

moduli of elasticity of concrete and steel was approximately fifteen to one and therefore, for an equivalent length, the moments of inertia need be only one fifteenth the value required by similitude. The final section of the steel strut (see Fig. 3) used was 1.315 in. by 0.582 in.

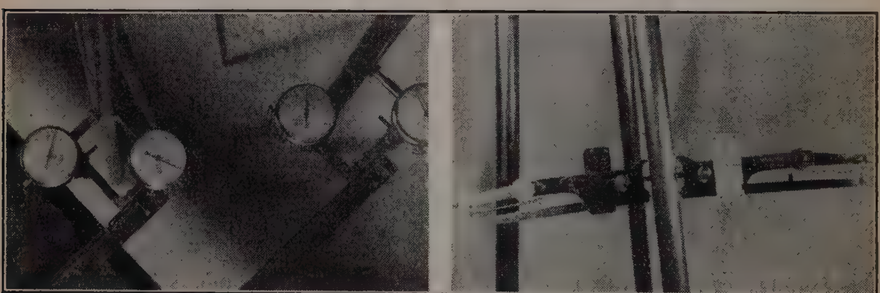
Pile Restraint

The condition that is most difficult to reproduce in the model is the resistance to lateral displacement of the piles offered by the material through which they are driven. This resistance is negligible at the mud line but increases as stiffer materials are encountered at lower elevations. The resistance that successive layers offer to lateral movements seldom varies uniformly, and it is almost impossible to simulate such a resisting medium satisfactorily. Attempts have been made to duplicate such passive action by using similar materials of varying densities and degrees of compaction, as well as purely mechanical devices (such as a series of springs distributed along the pile length and adjusted to represent the supposed variation in resistance). In view of these difficulties, no attempt was made to simulate such restraint.

As constructed, the model represents a system of piles fixed at their bases, restrained by the pier cap, and unrestrained throughout their length, which is, perhaps, the most unfavorable condition of restraint. The stresses so determined should give a fair indication of the maximum stresses to be expected in the prototype.

Measuring Devices

The deflection gages shown in Fig. 4(*a*) were graduated to read in thousandths of an inch and had a maximum travel of about $\frac{3}{8}$ in. Two gages were



(*a*) (*b*)

Fig. 4.—Testing Instruments

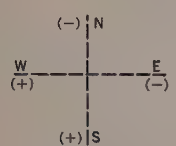
applied simultaneously at a given point with their axes parallel to the transverse and longitudinal axes of the pier and lying in a horizontal plane. The resulting measurements indicated the movements of that particular point in two perpendicular directions in a horizontal plane.

Two tensometers (see Fig. 4(*b*)), each having a multiplication factor of 1,075, and with 8-in. extensions to further magnify the extremely small strains, were used for the strain measurements. These gages were applied simultaneously at opposite ends of a diameter in either the transverse or longitudinal direction. Readings were taken in this position, and then the two tensometers were applied and read in the perpendicular direction.

Applied Loads

The direct load (dead, live, and impact) used in the tests was 600 lb, which corresponds to a load on the prototype of 993,420 lb. The applied wind load was 60 lb, being the equivalent of a wind force of 99,340 lb on the trusses. The allowance to be made for ice thrust is quite arbitrary, and in these tests the transverse and longitudinal ice loads were each represented by a force of 120 lb, which corresponds to a thrust on the prototype of 198,680 lb.

TABLE 2.—Location of Test Points

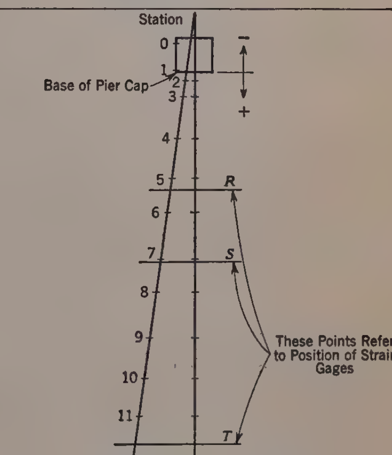


(*a*) SIGN OF DEFLECTIONS

(*b*) PLAN OF PIER CAP

	Distance of Point From Base of Pier Cap, in Inches	
	Battered Pile	Vertical Pile
0[a]		$-3\frac{5}{8}$
1[a]		$-\frac{1}{8}$
Base of Pier Cap		
2	1	1
3	$3\frac{1}{8}$	$3\frac{1}{8}$
4	$8\frac{3}{8}$	$8\frac{1}{4}$
5	$13\frac{1}{2}$	$13\frac{1}{4}$
R	15	$14\frac{3}{4}$
6	$17\frac{3}{4}$	$17\frac{1}{2}$
7	$23\frac{5}{8}$	$23\frac{1}{4}$
S	24	$23\frac{5}{8}$
8	28	$27\frac{5}{8}$
9	$33\frac{3}{4}$	$33\frac{1}{4}$
10	$38\frac{3}{4}$	$38\frac{1}{4}$
11	$43\frac{1}{2}$	$42\frac{7}{8}$
T	47	$46\frac{3}{8}$
Base of Piles		

[a] These Points Are on the Face of the Pier Cap



(*c*) PART ELEVATION OF PILES

The direct load was applied through a steel A-frame (Fig. 3), fixed at one end and freely supported at the other, and the wind and ice loads were applied by means of cables. These load-applying devices were kept constantly alined and were gradually adjusted to their proper tensions by calibrated scales.

Test Points and Loadings

Table 2 indicates the position of the test points. Deflections were measured at stations 0 to 11, and strain measurements were taken at stations R, S, and T. The deflections of the pier cap and the piles due to the four individual effects (direct load, wind load, and two mutually perpendicular ice

loads), and to a combination of loads, were observed and recorded. The combination loading represents the simultaneous application of the transverse ice, wind, and direct loads. Deflections and strain measurements were taken as in the case of the individual loads and were compared with the values obtained by adding the individual effects. The object of this comparison was to obtain an idea of the magnitude of the error involved if the secondary effects due to superposition were neglected.

For purposes of brevity, this paper includes only the data and necessary computations for the east-west and the north-south ice loads (east-west is the longitudinal axis of the bridge and north-south the transverse axis). These conditions were selected because the deflections and stresses are of reasonable magnitude and they best illustrate the action of the pier: (1) As two cantilever units (under the east-west ice load); and (2) as a portal (north-south ice load).

Development of the Elastic Curve

The deflections of the various elements of the model were measured for two purposes: (1) To predict the deflections of the prototype pier cap; and (2) to develop the equation of the elastic curve of each pile and, by use of the well-known formula

$$M = E\,I\,\frac{d^2y}{dx^2} \qquad (1)$$

to predict the end moments set up by such deflections. As a check, these end moments will be compared with those determined from the strain measurements.

Any curve, such as the deflection curve of a structural member subject to lateral loading or to applied end moments, may be represented by a cubic equation of the form

$$y = a\,x + b\,x^2 + c\,x^3 \qquad (2)$$

This general equation, although not precise, was used in the analysis because of its simplicity and the accuracy with which the resulting curves agree with the measured deflections. In view of the number of unknown coefficients, Eq. 2 could be solved with a minimum of three sets of coordinates. However, three points would not be sufficient to represent the actual deflected position with any degree of accuracy, and it was decided to use a greater number of points—nine, in this case.

In order to evaluate the coefficients, the coordinates (x, y) of each point are substituted in the general equation, as follows:

$$y_1 = a\,x_1 + b\,x_1^2 + c\,x_1^3 \qquad (3a)$$

$$y_2 = a\,x_2 + b\,x_2^2 + c\,x_2^3 \qquad (3b)$$

$$y_3 = a\,x_3 + b\,x_3^2 + c\,x_3^3 \qquad (3c)$$

$$\cdots\cdots\cdots$$

$$y_n = a\,x_n + b\,x_n^2 + c\,x_n^3 \qquad (3n)$$

If Eqs. 3 are added:

$$\Sigma y = a \Sigma x + b \Sigma x^2 + c \Sigma x^3 \qquad (4a)$$

Furthermore, if Eqs. 3 are multiplied by x_1, x_2, x_3, etc., respectively, and again by x_1^2, x_2^2, x_3^2, etc., the result is two sets of equations which, when added, have the following general form:

$$\Sigma (xy) = a \Sigma x^2 + b \Sigma x^3 + c \Sigma x^4 \qquad (4b)$$

and,

$$\Sigma (x^2 y) = a \Sigma x^3 + b \Sigma x^4 + c \Sigma x^5 \qquad (4c)$$

Eq. 2 lacks precision, in that it fails to satisfy a primary condition. By differentiating and setting $x = 0$, $\frac{dy}{dx} = a$, which is obviously incorrect since the slope at the origin is zero, in view of the fact that the pile is fixed at this point. However, this slope is extremely small and Eq. 2 will be used since the corresponding curves fit the given points more closely than those obtained from some equations that satisfy this theoretical condition.

Each pile is assumed to have its maximum deflection, and consequent maximum bending stresses, in a plane which contains that pile and whose horizontal trace is parallel to the direction of the applied load. There are small deflections (Tables 3 and 4) in a direction perpendicular to that of the external loading and,

TABLE 3.—Deflection of Pier Caps, in Thousandths of an Inch (Positive Unless Otherwise Indicated)

Point	Part I.—North-South Ice				Part II.—East-West Ice			
	Station 0		Station 1		Station 0		Station 1	
	North-south	East-west	North-south	East-west	East-west	North-south	East-west	North-south
1	34.0	2.5	36.7	0.0	....	....	82.5	−2.3
2	34.5	−0.8	36.4	−0.2	....	....	79.0	1.0
3	35.0	−1.8	36.9	−0.4	56.7	−1.1	80.1	0.5
4	35.2	1.3	37.1	2.4	57.8	−1.7	81.0	−2.0

theoretically, these should be resolved with the former in order to arrive at the maximum values indicated by test. However, in all cases of lateral loading, the small deflections that accompany the much larger deflections in the plane of loading are less than 10% of the latter and are neglected. The neglect of these minor effects may be justified by noting that the resultant of two mutually perpendicular forces, one of which is 10% of the other, is less than 1% greater than its largest component. (In some cases, deflection readings of the pier caps, Table 3, could not be taken because either the piles or the loading devices interfered with the placing of the gages.)

The equation of the elastic curve of pile 2 for the east-west ice load will be developed for purposes of illustration. The deflection curve is plotted in the

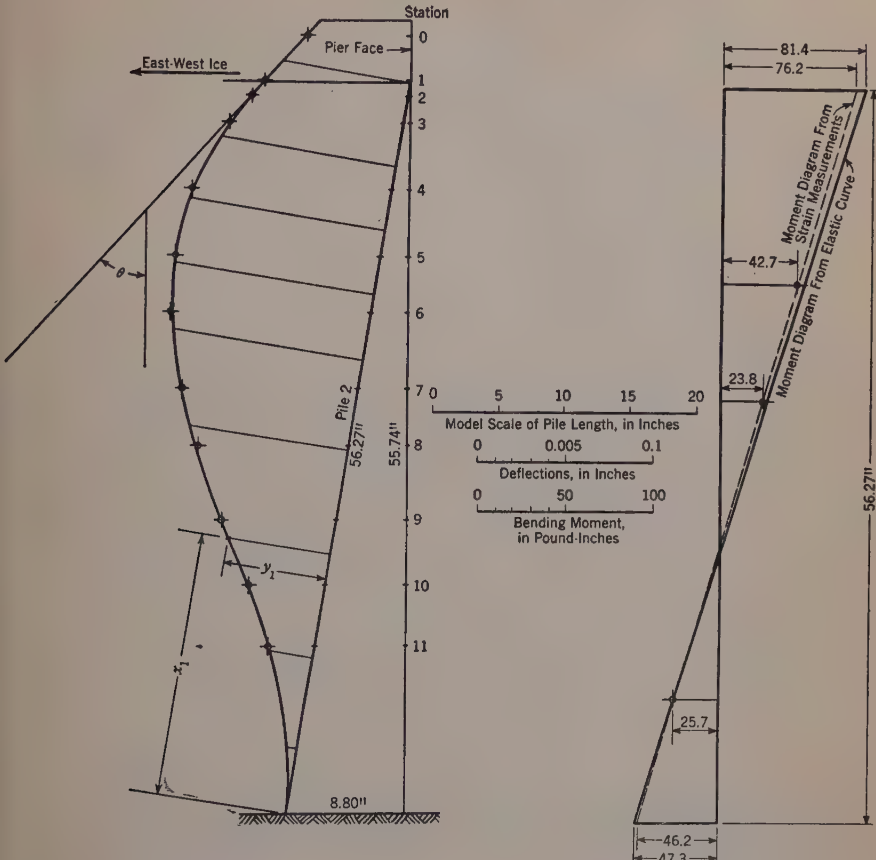
plane of the pile's deflection as shown in Fig. 5. Each battered pile appears as such, its indicated length is its true length, and the deflections are plotted, as

TABLE 4.—Deflections of Piles, in Thousandths of an Inch (Positive Unless Otherwise Indicated)

Pile	Deflection at Station (See Table 2):									
	2	3	4	5	6	7	8	9	10	11
	(a) Part I.—North-South Ice; North-South Deflections									
1	38.0	38.5	37.4	34.2	30.6	23.9	18.6	12.7	7.6	4.0
2	37.2	37.6	36.6	34.0	30.8	25.7	20.9	15.5	10.1	6.2
3	36.1	36.3	35.9	33.0	29.1	24.9	19.8	14.1	9.1	5.5
4	37.2	38.2	37.0	35.1	32.6	27.7	22.8	17.1	11.6	7.1
5	37.2	38.0	37.0	35.7	32.7	28.2	24.0	17.8	12.9	7.9
6	36.5	37.0	34.9	32.2	28.7	24.0	19.2	13.4	8.6	5.1
7	36.9	37.7	35.7	32.5	28.8	23.2	18.4	13.3	9.2	5.8
8	37.4	38.1	38.0	35.0	31.8	26.2	21.5	15.0	10.0	5.7
9	....	....	....	34.0	30.7	25.0	20.3	14.0	9.3	5.7
10	....	....	....	33.8	30.0	24.3	19.0	14.0	9.2	5.8
	(b) Part I.—North-South Ice; East-West Deflections									
1	0.4	0.6	0.7	−1.0	−0.3	−1.5	−1.9	−1.9	−1.1	−1.1
2	0.7	0.9	−0.1	−0.9	0.0	0.0	−0.3	−0.3	0.0	−0.4
3	0.3	0.3	−0.2	−2.7	−2.1	−4.0	−4.0	−4.1	−3.0	....
4	0.2	0.8	0.2	−0.3	0.9	0.5	0.5	0.2	0.7	0.5
5	0.4	0.0	−0.9	0.0	−0.1	−0.5	−0.5	−0.9	−1.0	−0.7
6	1.5	1.2	0.6	−0.5	−1.8	−2.6	−3.0	−2.6	−2.0	−1.3
7	0.3	2.0	2.1	2.0	0.6	1.8	1.0	0.2	0.0	....
8	1.2	2.6	1.7	1.6	1.8	2.2	1.8	1.1	1.2	1.0
9	....	....	....	....	0.6	1.0	0.0	0.0	0.2	0.0
10	....	....	....	−0.3	−0.3	0.0	0.0	−0.2	0.0	0.2
	(c) Part II.—East-West Ice; North-South Deflections									
1	0.0	0.9	−1.4	1.0	−2.4	−0.6	−1.1	−1.0	0.0	−0.1
2	−1.9	−3.0	−1.9	−0.7	−3.6	−2.4	−1.8	−1.6	−1.0	−0.9
3	0.8	0.3	0.0	4.4	1.4	0.0	0.0	1.1	1.2	0.0
4	0.8	−1.4	−2.1	0.0	−4.7	−3.5	−3.0	−2.4	−2.0	−1.9
5	−0.7	0.0	1.7	−2.1	−0.5	0.2	1.1	2.0	2.6	1.0
6	−2.0	−2.0	−4.6	−2.8	−0.7	−0.8	0.4	0.0	1.0	0.0
7	2.0	−1.8	−2.0	−2.0	0.0	−2.1	−1.6	−0.3	0.5	−0.3
8	−2.3	−4.9	−3.7	−5.1	−3.7	−2.6	−3.1	−0.7	−0.9	−1.0
9	....	....	....	....	1.0	0.1	1.2	0.8	0.7	0.4
10	....	....	....	0.0	0.3	−1.7	−0.6	0.0	0.0	−0.4
	(d) Part II.—East-West Ice; East-West Deflections									
1	88.7	99.0	115.8	121.1	118.3	105.1	90.7	67.8	47.0	29.0
2	87.0	97.1	112.4	116.0	112.5	98.5	84.3	63.9	43.1	26.1
3	87.3	98.0	112.3	115.1	111.2	97.3	81.9	60.0	41.7	....
4	87.6	98.2	115.0	120.4	116.9	104.1	87.8	64.9	44.3	27.8
5	87.3	98.0	113.5	116.9	113.0	99.4	84.4	62.2	43.1	26.9
6	87.9	97.8	111.1	114.0	110.1	95.8	81.1	59.5	41.9	24.9
7	86.2	95.9	110.1	112.8	108.0	95.3	81.6	60.1	42.2	....
8	87.6	97.8	112.8	115.3	111.4	97.2	83.2	62.0	43.9	26.8
9	....	....	....	114.4	106.6	93.6	79.4	58.2	39.7	23.7
10	....	....	....	115.1	110.5	97.0	81.9	60.4	42.0	25.0

measured, horizontally. The points representing the displacement of the pier cap are the deflections of the center of the south cap as determined by averaging the deflections of points 1 and 4 (Table 3).

In developing the equation of the deflected pile, convenient abscissas are located along the pile axis, and the corresponding ordinates, in thousandths of an inch, are measured perpendicular to that axis. These coordinates appear in



(*a*) DEFLECTION CURVE
(In Plane of Pile)

(*b*) MOMENT DIAGRAM

FIG. 5.—TYPICAL DEFLECTION CURVE AND CORRESPONDING MOMENT DIAGRAM

the first and second columns of Table 5, the remaining columns being self-explanatory. These summations are substituted in Eqs. 4, as follows:

$$684.0 = 291.27\,a + 11{,}869\,b + 532{,}973\,c \quad \text{(5a)}$$

$$26{,}589 = 11{,}869\,a + 532{,}973\,b + 25{,}232{,}786\,c \quad \text{(5b)}$$

and

$$1{,}129{,}171 = 532{,}973\,a + 25{,}232{,}786\,b + 1{,}236{,}748{,}279\,c \quad \text{(5c)}$$

Then, solving simultaneously, the following expression represents the elastic curve of pile 2 as deflected by the east-west ice load:

$$y = 0.3989\,x + 0.1737\,x^2 - 0.002803\,x^3 \quad \text{(6)}$$

TABLE 5.—Development of the Equation of the Deflected Pile

y	x	x^2	x^3	x^4	x^5	$x\,y$	$x^2\,y$
0.0	0.00	0	0	0	0	0	0
5.6	5.00	25	125	625	3,125	28	140
25.0	12.00	144	1,728	20,736	248,832	300	3,600
55.2	20.00	400	8,000	160,000	3,200,000	1,104	22,080
86.4	28.00	784	21,952	614,656	17,210,368	2,419	67,738
107.0	35.00	1,225	42,875	1,500,625	52,521,875	3,745	131,075
115.0	40.00	1,600	64,000	2,560,000	102,400,000	4,600	184,000
113.5	45.00	2,025	91,125	4,100,625	184,528,125	5,108	229,838
101.3	50.00	2,500	125,000	6,250,000	312,500,000	5,065	253,250
75.0	56.27	3,166	178,168	10,025,519	564,135,954	4,220	237,450
684.0	291.27	11,869	532,973	25,232,786	1,236,748,279	26,589	1,129,171

The second derivative of this equation is

$$\frac{d^2y}{dx^2} = 0.3474 - 0.016818\,x \quad \text{.....................(7)}$$

and, if this is substituted in Eq. 1, the moment can be determined at any point along the pile axis. Thus, since $M = \frac{30{,}000{,}000 \times 0.004525}{1{,}000} \frac{d^2y}{dx^2} = 135.75\,(0.3474 - 0.016818\,x)$, the two end moments are as follows: For $x = 0$, $M = 135.75\,(0.3474) = 47.16$ lb-in., and, for $x = 56.27$, $M = 135.75\,(-0.5989) = -81.30$ lb-in. (In the foregoing solution, 1,000 is introduced in the denominator because, throughout this analysis, the deflections have been measured in thousandths of an inch.) The resultant shear is $S = \frac{47.16 + 81.30}{56.27} = 2.28$ lb, and the equivalent horizontal shear, obtained by a comparison of similar triangles (Fig. 2), is $S_h = \frac{2.28 \times 63.46}{64.33} = 2.25$ lb.

Strains

Strain measurements were taken at three principal levels located at 15 in., 24 in., and 47 in. below the base of the pier cap (see Table 2). The object of these measurements was to determine the axial load and bending moments in each pile, the former remaining constant throughout but the latter varying and, in view of the absence of lateral loads on the pile, having maximum values at its extremities. It was obviously impossible to measure the deformations at the end of each pile, but the gages were applied as close to the ends as physical obstructions and convenience of reading permitted. The middle point was located so as to lie above the point of contraflexure in most cases.

The algebraic sum of the measured strains at the ends of a diameter, multiplied by a suitable constant, will give the axial load in the pile, and the algebraic difference of these strains, multiplied by another constant, will give the moment in the pile, in the plane of the gages, at a point corresponding to the center of the gage length. Strain readings in a plane perpendicular to this will yield moments, which, when resolved with the first set, will give the maximum values. Only the moments in the plane of the pile's deflection were considered because,

as in the case of the deflections, the moments in the plane perpendicular to this were sufficiently small to neglect without appreciably affecting the indicated maxima.

A typical set of strain readings is reproduced herein, as well as the method of computing the axial loads and the corresponding moments:

								Average strain
		0.434		0.430		0.438		−0.434
	0.410	0.457	0.407	0.453	0.431	0.445		
N-gage	0.880	0.470	0.927	0.520	0.973	0.542	0.987	
		0.738		0.735		0.735		−0.736
	0.718	0.758	0.712	0.757	0.730	0.740		
S-gage	1.205	0.487	1.245	0.533	1.290	0.560	1.300	

In this instance the gages were located in a north-south plane at station T, pile 1, and the structure was deflected by the north-south ice load. In each series of readings an average of the differences obtained by applying and removing the load was considered a single strain measurement. Three consistent readings (such as the foregoing) were considered satisfactory. The values of the constants previously mentioned can be determined by a consideration of the following expressions which represent the stress condition at opposite ends of a diameter:

$$S_n = \frac{P}{A} + \frac{M\,c}{I} \qquad (8a)$$

and

$$S_s = \frac{P}{A} - \frac{M\,c}{I} \qquad (8b)$$

If these equations are added:

$$\frac{P}{A} = \frac{S_n + S_s}{2} \qquad (9)$$

But

$$S_n = E\,\delta_n = E \times \frac{\Delta_n}{8 \times 1{,}075} \qquad (10a)$$

and

$$S_s = E\,\delta_s = E \times \frac{\Delta_s}{8 \times 1{,}075} \qquad (10b)$$

in which Δ is the total deformation over an 8-in. gage length as recorded with a gage magnification of 1,075; and δ = unit deformation. Substituting Eqs. 10 in Eq. 9 and denoting $(\Delta_n + \Delta_s)$ by Δ':

$$P = \frac{A\,E}{8 \times 1{,}075} \times \left(\frac{\Delta_n + \Delta_s}{2}\right) = \frac{A\,E}{17{,}200} \times \Delta' = 119.4\,\Delta' \qquad (11)$$

Similarly, subtracting Eq. 8*b* from Eq. 8*a*:

$$\frac{M\,c}{I} = \frac{S_n - S_s}{2} \qquad (12)$$

Substituting Eqs. 10 in Eq. 12 and denoting $(\Delta_n - \Delta_s)$ by Δ'':

$$M = \frac{I}{c} \times \frac{E}{8 \times 1{,}075} \times \left(\frac{\Delta_n - \Delta_s}{2}\right) = \frac{E\,I}{17{,}200\,c} \times \Delta'' = 20.87\,\Delta'' \qquad (13)$$

Thus, in the case of the foregoing illustrative example, the axial load and the

TABLE 6.—LOADS AND MOMENTS DERIVED FROM MEASURED STRAINS

Pile No.	Part I.—North-South Ice										Part II.—East-West Ice		
	Axial Loads, in Pounds							Average Moments, in Pound-Inches at Stations:					
	Station R		Station S		Station T		Average load	R	S	T	R	S	T
	North-south	East-west	North-south	East-west	North-south	East-west							
1	140.0	143.0	141.2	141.6	139.7	141.3	−141.1	−7.2	−1.3	6.3	−46.5	−26.8	26.7
2	107.9	108.2	107.0	109.4	109.1	107.5	+108.2	−5.9	−2.3	6.2	−42.7	−23.8	25.7
3	167.9	168.5	166.4	167.2	168.8	170.4	−168.2	−5.5	−2.0	6.4	−43.3	−21.5	26.3
4	94.8	93.9	92.2	95.3	92.4	93.4	+ 93.7	−5.9	−3.4	6.8	−45.9	−26.6	27.9
5	132.9	132.5	137.2	135.6	134.5	137.1	+135.0	−5.5	−3.5	6.4	−43.1	−22.6	25.2
6	110.6	110.8	112.0	110.1	111.0	110.8	−110.9	−5.3	−1.8	6.1	−41.5	−22.1	24.9
7	164.5	162.1	163.6	163.8	163.2	162.6	+163.3	−4.3	−0.8	5.1	−39.8	−20.0	23.0
8	86.6	91.2	90.3	89.1	90.2	87.6	− 89.2	−7.0	−2.9	6.6	−42.2	−21.9	24.2
9	8.3	8.6	8.8	7.9	8.8	8.1	+ 8.4	−4.7	−1.7	5.5	−40.9	−19.5	25.0
10	7.3	8.4	6.0	6.2	9.3	5.7	− 7.1	−4.7	−1.4	5.4	−42.7	−21.5	26.7

TABLE 7.—AXIAL LOADS, IN POUNDS, DERIVED FROM MEASURED STRAINS; Part II.—East-West Ice

Pile No.	North-South					East-West				Average of north-south and east-west averages	Percentage variation based on average
	Determinations				Average	Determinations			Average		
1	83.3 85.5 88.6	84.0 86.4 91.0	84.0 88.6	85.2 88.6	86.6	95.3 97.1 101.9	96.3 98.9	96.3 100.0	98.0	− 92.3	±6.2
2	129.2	131.3	132.6	....	131.0	130.4	131.3	133.0	131.6	−131.3	±0.2
3	77.4	79.8	83.8	....	80.3	80.5	81.7	....	81.1	− 80.7	±0.5
4	129.4 135.7	130.6	131.9	134.7	132.5	139.7 142.5	140.2 143.3	141.4	141.4	−137.0	±3.3
5	94.1	95.5	97.5	....	95.7	95.0 101.5	97.7 101.5	97.7	98.7	+ 97.2	±1.5
6	115.0	118.2	119.0	120.6	118.2	124.0	124.0	125.2	124.4	+121.3	±2.6
7	79.7 82.6	80.2	81.0	82.6	81.2	87.2 89.8	87.4 89.8	87.4	88.3	+ 84.8	±4.2
8	124.2 131.0	125.7 131.8	126.4	127.5	127.8	128.7 133.8	129.0	131.4	130.7	+129.3	±1.2
9	2.2 8.4	3.6 10.5	4.3	8.1	6.2	4.8	5.5	6.4	5.6	+ 5.9	....
10	+2.4 −0.5	+1.2 −1.9	+0.4 −2.4	0.0 −3.6	−0.6	−1.9 0.0	−1.4 0.0	−1.2 +1.2	−0.5	−0.5	....

bending moment are as follows: $P = 119.4\,[(-0.434) + (-0.736)] = -139.7$ lb; and $M = 20.87\,[(-0.434) - (-0.736)] = +6.30$ lb-in.

It would be impracticable to tabulate all of the recorded strains, but the resulting axial loads and bending moments, computed in the foregoing manner, are listed in Tables 6 and 7. The column headings for the axial loads indicate

TABLE 8.—COMPARISON OF MOMENTS, IN POUND-INCHES

Pile No.	Part I.—North-South Ice				Part II.—East-West Ice			
	Top Moments		Base Moments		Top Moments		Base Moments	
	From elastic curve	From strain measurements	From elastic curve	From strain measurements	From elastic curve	From strain measurements	From elastic curve	From strain measurements
1	−13.0	−12.7	12.5	11.9	−84.1	−81.2	47.5	48.0
2	− 9.6	−11.3	8.4	9.5	−81.4	−76.2	47.3	46.2
3	−11.1	−11.1	9.8	9.8	−78.0	−76.0	46.4	47.0
4	−10.8	−11.5	8.5	9.3	−84.7	−82.3	48.2	49.8
5	−10.4	−10.7	7.5	8.4	−67.8	−74.8	42.0	45.4
6	−11.5	−10.4	10.5	9.5	−63.8	−72.0	41.2	45.0
7	− 6.8	− 8.4	7.1	8.0	−63.9	−68.6	39.5	42.0
8	−13.2	−13.3	10.9	10.6	−63.6	−72.8	40.4	44.2
9	−11.5	− 9.5	10.2	8.5	−73.4	−71.5	48.8	46.0
10	−11.3	− 9.2	10.7	8.7	−77.0	−75.0	49.7	47.0
Average	−10.9	−10.8	9.6	9.4	−73.8	−75.0	45.1	46.1

the plane of the gages during the various determinations, and the headings for the average moments refer to the points at which the pile moments in the planes of loading were measured. In compiling Table 7, relatively large discrepancies were observed between indicated axial loads in the two planes of measurement. A large number of readings taken at various points along the pile were necessary

TABLE 9.—SUMMATION OF VERTICAL AND HORIZONTAL FORCES, IN POUNDS

Pile No.	Part I.—North-South Ice					Part II.—East-West Ice				
	Axial load	Vertical component	Horizontal Component		Horizontal shear due to bending	Axial load	Vertical component	Horizontal Component		Horizontal shear due to bending
			East-west	North-south				North-south	East-west	
1	−141.1	−139.2	−12.85	19.28	0.43	− 92.3	− 91.0	12.62	8.42	2.28
2	108.2	106.7	14.78	9.85	0.37	−131.3	−129.5	−11.97	17.95	2.19
3	−168.2	−165.9	−15.32	22.98	0.38	− 80.7	− 79.6	11.03	7.36	2.17
4	93.7	92.4	12.80	8.53	0.37	−137.0	−135.1	−12.49	18.73	2.36
5	135.0	133.2	−12.30	16.44	0.34	97.2	95.9	13.28	8.86	2.12
6	−110.9	−109.4	15.15	10.10	0.36	121.3	119.6	−11.06	16.58	2.09
7	163.3	161.1	−14.87	22.31	0.30	84.8	83.6	11.58	7.73	1.96
8	− 89.2	− 88.0	12.19	8.12	0.43	129.3	127.5	−11.79	17.67	2.09
9	8.4	8.4	0.0	0.0	0.32	5.9	5.9	0.0	0.0	2.12
10	− 7.1	− 7.1	0.0	0.0	0.32	− 0.5	− 0.5	0.0	0.0	2.20
Summation		− 7.8	− 0.42	119.61	3.62	....	− 3.2	+ 1.20	103.30	21.58
Percentage discrepancy		± 0.8	± 0.4	....	+2.7	....	± 3.7	± 1.3	....	+4.1

to obtain a fair average. The end moments obtained by extrapolation from these tabulated values are compared in Table 8 with those determined from the elastic curve.

TABLE 10.—SUMMATION OF MOMENTS (IN POUND-INCHES) ABOUT THE x-AXIS, IN THE y z PLANES

Pile No.	Part I.—North-South Ice					Part II.—East-West Ice		
	Moment at base of pile (1)	Vertical component of axial load (2)	Distance from x-axis, in inches (3)	Col. 2 times Col. 3 (4)	Total moment; Col. 1 plus Col. 4 (5)	Vertical component of axial load (6)	Distance from x-axis, in inches (7)	Moment; Col. 6 times Col. 7 (8)
1	12	139.2	−15.30	2,130	2,142	− 91.0	−15.30	1,392
2	9	106.7	− 0.63	− 67	− 58	−129.5	− 0.63	82
3	10	−165.9	− 2.30	382	392	− 79.6	− 2.30	183
4	9	92.4	12.37	1,143	1,152	−135.1	12.37	1,671
5	8	133.2	15.30	2,036	2,044	95.9	15.30	1,467
6	9	−109.4	0.63	− 69	− 60	119.6	0.63	75
7	8	161.1	2.30	370	378	83.6	2.30	192
8	11	− 88.0	−12.37	1,089	1,100	127.5	−12.37	1,577
9	8	8.4	− 6.50	− 55	− 47	5.9	− 6.50	− 38
10	9	− 7.1	6.50	− 46	− 37	− 0.5	6.50	− 3
Sum	+93	....	....	+6,913	+7,006	....	....	+ 102
Moment on pier (= 120 lb × 56 in.)				....	−6,720	....	....	....
Percentage discrepancy $\left(=\frac{7{,}006 - 6{,}720}{6{,}720}\right)$				....	+4.3	....	....	±1.5

TABLE 11.—SUMMATION OF MOMENTS (IN POUND-INCHES) ABOUT THE y-AXIS, IN THE x z PLANES

Pile No.	Part I.—North-South Ice			Part II.—East-West Ice				
	Vertical component of axial load (1)	Distance from y-axis, in inches (2)	Moment; Col. 1 times Col. 2 (3)	Moment at base of pile (4)	Vertical component of axial load (5)	Distance from y-axis, in inches (6)	Col. 5 times Col. 6 (7)	Total moment; Col. 4 plus Col. 7 (8)
1	−139.2	−5.87	817	47	− 91.0	−5.87	534	581
2	106.7	−8.80	−939	46	−129.5	−8.80	1,140	1,186
3	−165.9	−5.87	974	46	− 79.6	−5.87	467	513
4	92.4	−8.80	−813	49	−135.1	−8.80	1,189	1,238
5	133.2	5.87	781	45	95.9	5.87	563	608
6	−109.4	8.80	−963	44	119.6	8.80	1,052	1,096
7	161.1	5.87	946	41	83.6	5.87	491	532
8	− 88.0	8.80	−774	44	127.5	8.80	1,122	1,166
9	8.4	0.0	0	45	5.9	0.0	0	45
10	− 7.1	0.0	0	46	− 0.5	0.0	0	46
Sum	....	....	+ 29	+453	....	....	+6,558	+7,011
Moment on pier			....	(= 120 lb × 56 in.)				−6,720
Percentage discrepancy			±0.4	$\left(=\frac{7{,}011 - 6{,}720}{6{,}720}\right)$				+4.3

Tables 9, 10, 11, and 12 are included as a general check on the validity of the experimental determinations of the axial loads and bending moments based on strain measurements. Table 9 represents the summation of the vertical

and horizontal forces; that is, the three spatial components of the axial loads. For example, in the case of the north-south ice load, where the applied load is horizontal and in a north-south direction, the sum of the vertical components should be zero, the sum of the east-west horizontal components should be zero, and the sum of the north-south horizontal components, when combined with the horizontal shears due to bending, computed as shown in the text at Eqs. 6 and 7, should equal the applied load of 120 lb. In this case, the agreement of the test

TABLE 12.—SUMMATION OF MOMENTS (IN POUND-INCHES) ABOUT THE z-AXIS, IN THE $x\,y$ PLANE AT THE BASE

Pile No.	PART I.—NORTH-SOUTH ICE							PART II.—EAST-WEST ICE				
	Moment of North-South Components			Moment of East-West Components			Total; Col. 3 plus Col. 6	Moment of North-South Components		Moment of East-West Components		Total; Col. 9 plus Col. 11
	North-south component	Distance from z-axis, in inches	Moment; Col. 1 times Col. 2	East-west component	Distance from z-axis, in inches	Moment; Col. 4 times Col. 5		North-south component	Moment; Col. 8 times Col. 2	East-west component	Moment; Col. 10 times Col. 5	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	19.28	−5.87	−113	−12.85	−15.30	197	84	12.62	− 74	− 8.42	129	55
2	9.85	−8.80	− 87	14.78	− 0.63	− 9	− 96	−11.97	105	−17.95	11	116
3	22.98	−5.87	−135	−15.32	− 2.30	35	−100	11.03	− 65	− 7.36	17	− 48
4	8.53	−8.80	− 75	12.80	12.37	158	83	−12.49	110	−18.73	−232	−122
5	18.44	5.87	108	−12.30	15.30	−188	− 80	13.28	78	− 8.86	−136	− 58
6	10.10	8.80	89	15.15	0.63	10	99	−11.06	− 97	−16.58	− 10	−107
7	22.31	5.87	131	−14.87	2.30	− 34	97	11.58	68	− 7.73	− 18	50
8	8.12	8.80	72	12.19	−12.37	−151	− 79	−11.79	−104	−17.67	219	115
9	0.0	0.0	0	0.0	− 6.50	0	0	0.0	0	0.0	0	0
10	0.0	0.0	0	0.0	6.50	0	0	0.0	0	0.0	0	0
Sum	....	....	....	....	....	....	+ 8	....	....	....	....	+ 1
Percentage discrepancy				....	....	....	+1.1	....	....	....	....	±0.1

data with these requirements is indicated by the following: The discrepancy in the summation of the north-south horizontal forces equals $\frac{123.23 - 120}{120}$ $= + 2.7\%$.

The discrepancies in the other two directions are based on the variation from the average of the plus and minus totals, as shown in the following computations:

Discrepancy in summation of

$$\text{Vertical components} = \frac{\pm 3.9}{505.7} = \pm\, 0.8\%$$

$$\text{East-west horizontal components} = \frac{\pm\, 0.21}{55.13} = \pm\, 0.4\%$$

Tables 10, 11, and 12 indicate the summation of moments, of the axial load components and the base moments, about the x, y, and z axes, respectively. The base moments listed in Table 8 are in the planes of deflection of the various

piles; and, for use in Tables 9, 10, 11 and 12, they must be corrected, by ratios indicated on Fig. 2, to give the corresponding moments in the vertical planes of summation. The discrepancies in the moment summations are computed in a manner similar to those in the load summations. Referring to Tables 10, 11, and 12, it is to be noted that all piles are considered cut at the pier base and the forces and moments act on the structure as shown in Fig. 6. Clockwise moments are assumed positive. Moments applied at the base of each pile are neglected except when adding the moments in the plane of loading (see Tables 11 and 12, Part I).

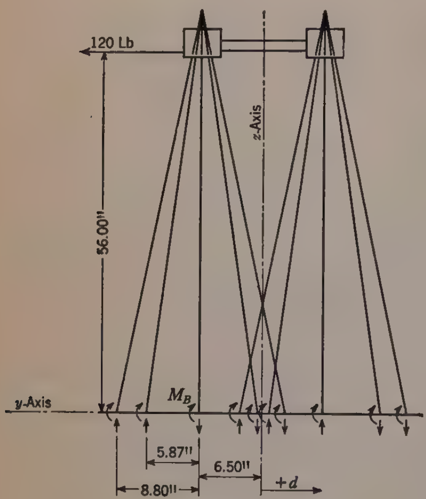


Fig. 6

Discussion of Data

The major deflections for each loading vary consistently and give smooth deflection curves as indicated by the typical curve in Fig. 5. When the pier is subjected to a lateral force, especially the east-west ice load, the deflections at right angles to the plane of loading vary with respect to magnitude and sign but are sufficiently small to neglect when investigating the behavior of the pier. The moments corresponding to these secondary deflections are extremely small and have not been included in any summations or comparisons. Tables 9, 10, 11, and 12 show that the agreement between the measured and applied axial loads and moments is satisfactory, the maximum error being approximately 4%.

However, in the case of the direct load, the test data for which have been omitted, the sum of the vertical components of the axial loads is only 569 lb, or approximately 5% less than the 600-lb applied load. This discrepancy might be due to errors in the load-applying device, the measuring gages, or possibly to some effect peculiar to this loading. Since the axial loads were of reasonable magnitude, and sufficient readings were taken to obtain a fair average, and since the scales in the loading device were checked carefully, this discrepancy might be due to the method of applying the direct load.

Summary of Test Results

In Table 13, the axial loads and maximum end moments in the model have been summarized for the five conditions tested. The maximum stresses have been computed from these values, using the familiar expression for combined direct and bending stress, $\frac{P}{A} \pm \frac{M\,c}{I}$.

Each of the axial loads listed in Table 13 is the average of six determinations (Tables 6 and 7), and each of these determinations is the result of at least three

consistent strain measurements as explained under the heading "Strains." The end moments in Table 13 were obtained from the moment curves established by strain measurements, these values being checked (see Table 8) by the moments based on the analytical treatment of the elastic curves as shown previously in the text. The end moments induced by direct loading (see Table 13, Part III) are negligible and were not included in the maximum stress computations.

TABLE 13.—TABULATION OF STRESSES

Pile No.	Part I.—North-South Ice			Part II.—Wind Load			Part III.—Direct Load	
	Axial load, in pounds	Maximum end moments, in pound-inches	Maximum stress, in pounds per square inch	Axial load, in pounds	Maximum end moments, in pound-inches	Maximum stress, in pounds per square inch	Axial load, in pounds	Maximum stress, in pounds per square inch
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	−141.1	±12.7	−3,123	−88.8	±5.1	−1,723	−55.2	−806
2	108.2	11.3	+2,525	17.3	4.8	+ 654	−56.7	−857
3	−168.2	11.1	−3,385	−56.4	2.3	−1,016	−63.5	−927
4	93.7	11.5	+2,329	75.3	2.8	+1,334	−54.3	−793
5	135.0	10.7	+2,866	104.3	2.8	+1,757	−52.7	−770
6	−110.9	10.4	−2,489	−27.6	2.4	− 605	−57.0	−832
7	163.3	8.4	+3,087	37.9	4.2	+ 905	−56.8	−830
8	− 89.2	13.3	−2,415	−70.1	5.8	−1,509	−58.0	−847
9	8.4	9.5	+ 917	−20.4	4.5	− 674	−59.1	−863
10	− 7.1	9.2	− 873	19.6	2.4	+ 487	−60.1	−878

TABLE 13.—(*Continued*)

Pile No.	Part IV.—Combination			Part V.—East-West Ice		
	Axial load, in pounds	Maximum end moments, in pound-inches	Maximum stress, in pounds per square inch	Axial load, in pounds	Maximum end moments, in pound-inches	Maximum stress, in pounds per square inch
	(9)	(10)	(11)	(12)	(13)	(14)
1	−300.0	±19.4	−6,003	− 92.3	±81.2	−8,136
2	72.1	16.6	+2,441	−131.3	76.2	−8,288
3	−298.6	12.0	−5,364	− 80.7	76.0	−7,532
4	114.5	12.5	+2,717	−137.0	82.3	−8,881
5	175.3	11.8	+3,546	97.2	74.8	+7,673
6	−198.1	11.6	−3,863	121.3	72.0	+7,791
7	144.5	13.0	+3,197	84.8	68.6	+6,972
8	−230.0	19.6	−4,997	129.3	72.8	+7,973
9	− 79.8	18.7	−2,728	5.9	71.5	+6,063
10	− 45.5	12.0	−1,668	− 0.5	75.0	−6,276

The moments and deflections as determined experimentally for the combination loading (Part IV, Table 13) do not differ appreciably from the sum of the individual effects similarly determined, probably because of the batter of the piles. It may be concluded that the secondary effects are negligible and, therefore, need not be considered in the analysis or design of the prototype.

By use of the conversion factors given in the text following Table 1, the maximum stress determined in the prototype is 28,158 lb per sq in. This stress

occurs in pile 1 when the pier is deflected by the least probable combination of loads (east-west ice, wind, and direct). The maximum lateral deflection determined in the prototype pier is 9.4 in. (east-west ice load) and the maximum longitudinal deflection is 6.2 in. (combination loading).

Acknowledgment

The complete record of data upon which this paper is based was submitted, by Mr. Coan, in 1938, as an essay to the School of Engineering, The Johns Hopkins University, Baltimore, Md., in conformity with the requirements for the degree of Master of Civil Engineering. A copy of the essay has been placed on file for reference in the Engineering Societies Library, 33 West 39th Street, New York, N. Y.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

THE PRACTICE OF STATE HIGHWAY DEPARTMENTS IN THE DESIGN OF ABUTMENTS

PROGRESS REPORT OF A SPECIAL SUBCOMMITTEE OF THE COMMITTEE OF THE STRUCTURAL DIVISION ON MASONRY AND REINFORCED CONCRETE

In 1938 the Committee issued a questionnaire (see Appendix) to the bridge engineers of the various state highway departments and of Hawaii and the District of Columbia. The purpose was to determine just how these departments are designing their abutments, what new types are being developed, what economies are being effected, what is being done in soil and foundation exploration at abutment sites, and how the information about the foundations is used after it is obtained. The questionnaire was an attempt to cover the field from the exploration of the site through the construction period.

The response to this study was most gratifying, only two failing to be returned. The Committee realized, of course, that every one is overburdened by questionnaires, but this seemed to be the only way to obtain the information. The very high percentage of replies, the evident care and thought given to the preparation of the answers, and the many letters of encouragement all indicate that bridge engineers are keenly interested in this matter of abutment design and realize that it is a very important phase of bridge construction. Much of the cost of short-span bridges is in the foundation. If difficulties with the bridge develop, they almost always can be attributed directly or indirectly to the foundation. Therefore, if any economies are to be effected, they can be done best by improving practice in the design and construction of the foundations. Edward W. Wendell, M. Am. Soc. C. E., of New York, states this very well in his letter on the questionnaire:

> "So much detail is advanced with reference to superstructure design and other structural activities that for a great many years I have felt that foundations were neglected. * * * For years I have been convinced that in too many instances throughout the country the foundations are left to the man in charge of construction, as an actual accomplishment in the field, to develop and change as he sees fit; in fact, the increases in cost of projects are traced almost invariably to a lack of proper recognition in the original foundation design."

In this discussion of the information obtained from the replies, the District of Columbia and Hawaii, for convenience, will be referred to as states.

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by October 15, 1940.

Exploration.—All the states indicate that they use some method of exploration of the foundation upon which the abutment will rest. Only one state indicated that it confines itself to one method—exploration by wash borings. All the other states use two or more methods, depending upon the conditions at the bridge site. Most of the experts on foundations and soil mechanics agree that the sounding rod is not a good tool for exploring a foundation and for determining the earth conditions at any considerable depth. There are bridge departments, however, whose engineers feel that the sounding rod is a very useful, as well as a very economical, tool, but it must be in the hands of an experienced man who has worked within the boundaries of a state for a long period. He must be a man who has probed many foundations, seen them opened up, and a structure placed on them. The sounding rod is the usual method of exploration in eight states, and twenty-six other states indicate more or less use of this method. When the probing is done with the special sampling tube, as developed and used by the California Highway Department,[1] excellent results are obtained and these soundings are very economical. Bridge departments should give serious consideration to the advisability of making soundings with the sampling tube.

The use of wash borings for exploring foundations is the usual procedure in twelve states. In thirteen more this method was checked as being used. In four of the states dry samples are taken in conjunction with this method and there may well be others since this question was not asked specifically. The replies would indicate that in the hands of an experienced man this is a good method even when dry samples are not taken. Of course, when dry samples are taken, the operator's judgment is largely eliminated and a great deal more confidence may be placed in the results.

The core drill is used for large and important structures or where the foundation conditions are unusually difficult. One state indicated it as the usual method. Thirty-three states indicated its use.

The auger is used by three states as the usual method and is given a varying amount of use by thirty-five more.

The test pit and loading platform are given occasional use by several states.

Four states indicate that an analysis of the soil at the bridge-site foundation is made. Four more departments do it occasionally. One state is organizing a soils division and three more departments are considering adopting this method. There is no indication in the replies as to how the information is to be used after it is obtained. There seems to be a willingness on the part of the state bridge engineers, either expressed or implied, to make soil analyses as soon as they see some method to apply the results of such an analysis to their designs to give better and more economical structures. Lester W. Millard, Assoc. M. Am. Soc. C. E., of Michigan, suggests that one result of the study should be some additional information on "the practical application of the theory of soil mechanics in determining safe loads on various soils."

Explorations by sounding rod, auger, and test pit are usually made by the regular survey parties, whereas explorations by wash borings, loading platforms,

[1] "Portable Equipment Used in Foundation Investigation," *Bulletin No. P-1*, State of California, Div. of Highways, Dept. of Public Works.

and core drillings are made by special survey parties, as they require more equipment. In two states the man in charge of the explorations is a geologist. Thirteen other states indicate that the man in charge of the parties is a specialist in the field of foundations. However, in these cases, the only special training that he has, other than a bachelor's degree, is "practical experience." In only two cases does the man in charge of this exploration work have a Master of Science degree, majoring in soils. The Subcommittee was surprized at this and believes that more use should be made of whatever the soils engineer has to offer.

Fill.—The fill behind the abutment and its proper drainage are an integral and very important part of the proper design and construction of abutments. The pressures developed are dependent upon the materials used and how they are placed. In this connection, N. R. Sach, of Missouri, states:

> " * * * one result of the investigation might be that of developing some procedure of making use of the actual soil conditions in determining the pressures to use in the design of abutments and retaining walls. In order to be of value, these pressures should be developed from results of actual soil study."

There is a similarity in thought between this quotation and that by Mr. Millard of Michigan.

The type of fill used behind a highway abutment is also very important from the point of view of its riding qualities. A highway bridge with a bump at each abutment cannot be considered entirely successful. Too often the fill material behind the abutment is nothing more than the unclassified material that occurs in the fill of the bridge approaches. Usually the fill material must also meet a specification that it will be "satisfactory to the engineer," which may mean much or very little. Gravel is required as the fill material behind the abutments by nine states. Eleven states select the material by a soil analysis. Nine states place the material, with the optimum water content, behind the abutments, but they did not make it clear in all cases that the section immediately behind the abutment is thoroughly drained. Three states require that special permeable fill material, placed with special care, be used behind the abutments. Indiana requires that the permeable fill material completely occupy the space from the backwall to the end of the wings and from there slope back, 1 on 3, to the natural ground. This permeable material is washed into place. On the other hand, its next-door neighbor, Illinois, allows no water to be used. The almost universal practice is to drain the fill immediately behind the abutment through weep holes. Water is led to these weep holes by various types of drains.

Design.—Abutments are analyzed exclusively by the Rankine theory of earth pressure by ten states. Twenty-four states use an equivalent liquid pressure exclusively. No state uses the Coulomb theory of earth pressure to the exclusion of all others, but one state uses it in the case of sloping ground or an inclined back to the abutment. Three states indicate that they use the Coulomb theory in conjunction with the Rankine theory, an equivalent liquid pressure, or a combination of all three. The remainder of the states use various combinations of the Rankine theory and an equivalent liquid pressure. That

these theories are all the same for the case of level fill and a vertical wall, or for a level fill and an assumed vertical plane at the heel, was noted by only one state. These replies check the conclusion that most retaining walls are designed by the Rankine theory in the United States, although most authorities now agree that Coulomb's theory approximates the actual conditions more closely.

The angle of repose of the earth is assumed to be approximately 33° by nine states. An equivalent liquid pressure of 30 lb is assumed by fifteen states. Nine states assume that the angle of repose or the equivalent liquid pressure varies from job to job and the variation is determined by judgment. In no case is the variation determined by a soil analysis.

The live load from the bridge is used as a superimposed load on the abutment by forty states and is not used by eight states. A live-load surcharge is used in back of the abutment by thirty-four states. There is a wide variation in the surcharge used, with the equivalent of 2 ft of fill being most common. The abutment is assumed to carry a traction, or braking load, by nineteen states, usually 10% of the live load. In eight states the live load from the bridge, a live-load surcharge, and a traction load are all applied to the abutment, and in three states none of them is used. This should make quite a difference in the design, other things being equal.

There is almost universal agreement that no water pressure should be assumed either in front or in back of the abutment. Some states indicated that they consider this as a possibility in each case.

The buried abutment, sometimes called spill-through, or skeleton abutment, is considered by most states to be designed, although two states frankly proportion such abutments by judgment. The structure is generally designed to withstand its own dead weight, the superimposed load, and the earth pressure on the back of the columns and cap of the abutment—that is, the columns and cap as seen in elevation from the rear of the abutment. As a general rule no allowance is made for the back pressure from the earth in front of the abutment. A few states make an allowance for the "drag" of the earth on the columns or for the arching of the earth between columns. This is done by increasing the equivalent liquid pressure or by multiplying the width of the columns by 1.5, or by some similar method. Harley G. Overholt, M. Am. Soc. C. E., of Minnesota, states: "We do not believe that the total overturning effect of the back fill over the entire length of the [buried] abutment is much reduced by the openings. The use of this type is probably overdone." The subcommittee is inclined to agree.

Types.—The question that aroused the most general disagreement was the one concerning the use of the buried or spill-through type of abutment. Some states are using it almost to the exclusion of all other types, whereas other states are almost vehement in their condemnation of its use. There are six states that indicate that more than 90% of their abutments are of this type: Washington in the west, Vermont in the northeast, Georgia and South Carolina in the southeast, Texas in the south, and Missouri in the middle west. E. B. Van de Greyn, of New Mexico, next-door neighbor to Texas, states:

"We do not favor skeleton or open type abutments except on overpass structures for railroad grade separation. For bridges over streams we consider them too expensive from the maintenance standpoint and dangerous to public due to flood waters washing fill, thus leaving a cavity under approach pavement, which cavity cannot be seen from roadway until roadway surface caves in."

In this connection Mr. Wendell states, for New York, that:

"So far as the buried, or skeleton type of abutment is concerned, I again wish to emphasize that we do not use them. My experience indicates that they are very unsatisfactory. They represent a false economy in that the first cost represents but a small percentage of the ultimate obsolescence. I have used them where we have the structure resting on rock, or on excellent hardpan, or where end-bearing piles can be driven to rock or hardpan. When this is done, all passive resistance must be taken up with batter piles."

On the other hand, B. J. Ornburn, of Montana, writes:

"It is noted that most of the questions in this inquiry pertain to solid wall abutments. Our experience leads us to declare that this is not the best type of construction to use in this section of the country. We use mostly open end bents with running fills and find that in the case of washouts we have very little damage done to the structure itself. The cost of replacing an approach fill is insignificant compared to the cost of replacing the structure. The only complete failure in the State on bridges built under our supervision was a structure that had semi-gravity type of abutments. An unusual flood occurred, and when the water could not obtain sufficient waterway area by washing out the fills, it scoured under the abutments and dropped them 4 to 6 ft. The writer [Mr. Ornburn] is a strong advocate of the use of the skeleton type abutment or end bent with curtain walls which gives the appearance of a solid wall abutment but has running fills and permits the washing out of approach fill in cloudburst sections instead of causing scour of stream bed."

Outside of New England and the northeastern states there are very few mass concrete abutments being constructed. There are a few exceptions. Michigan indicates that 25% of their bridges are of this type, Virginia 30%, Florida 40 to 50%, and the District of Columbia 40%. Elsewhere the notation is "5%," "rare," or "none." The semi-mass type, which had the further description in the questionnaire "gravity abutment with some reinforcement and pressure line outside middle third," is also popular in this northeastern section.

The cantilever type of abutment is very popular in the far west, whereas its use in any other section for more than 40% of a state's bridges is unusual.

The stub abutment, sometimes called sill abutment or bank block (that is, an abutment resting on piles and consisting of a bridge seat, backwall, and butterfly wings), is used by three states for 50% or more of their bridges, and occasionally by a few others. The practice seems to be to place the fill, compact it well, and then drive the piles through the fill. Paul B. Martin, of Kansas, writes:

"We have another type [stub abutment] which we have used with satisfactory results, especially on viaducts where there is no danger of

scour. We have found it necessary in the use of this type of abutment to take certain precautions, i.e., if the piling were to be driven through the fill material they should extend on down to point bearing on shale or rock. In some cases we have found this has required 80 ft or 90 ft steel piling. If rock or shale is not within reasonable distance we believe it to be a better practice not to drive the piling through the fill material. In this case, it is undesirable to use this type of abutment for continuous concrete structures.

"We have found that since we have required fills to be made of material containing the proper water content and compacted by means of sheepsfoot rollers they are so dense that it is sometimes impossible to drive timber piling through them. These heavy fills also have resulted in a settlement of the natural soil underneath the fill. To offset this we have required the top soil to be removed and the subsoil compacted as much as possible with the use of sheepsfoot rollers. We have also flattened our slopes somewhat to get a more stable embankment. As a result we see no serious objection and a great deal of economy in the use of either of these types of abutments [stub or buried]."

Most of the states are interested in improving the architecture of their abutments. There appears to be little agreement as to how this can be accomplished. The engineers feel that each structure is a separate problem. The general impression seems to be that the best effect is obtained by well-balanced proportions and simplicity of design, with a very moderate use of copings, flutings, and panels. Many bridge engineers seem to feel that considerable money is wasted in rubbing the parts of a structure which will never be seen by the general traveling public. More thought should be put on obtaining pleasing lines as it is the general sweep of the structure that catches the eye of the motorist as he flashes by.

The questions under "unsatisfactory experience" were not answered any too frankly in many instances. This was expected, however, as engineers are very loath to admit to any one outside their own personnel that any of their structures gives them any trouble. Several departments suggest that their unsatisfactory experience under the head of cracking is due to failure to use temperature steel, and others suggest that much trouble with abutments on piles may be eliminated by the use of batter piles. The bump at stub or spill-through abutments may be eliminated by using a berm in front of the bridge seat and plenty of well-compacted permeable fill in back of the abutment. The Subcommittee was left with the impression (even after allowance was made for the reluctance of engineers to report failures) that highway departments are designing abutments that are giving a very high order of satisfaction. However, the Committee was surprized at the large number of states (fifteen) which do not inspect their substructures systematically for approaching trouble.

Piles.—It is the universal policy of design departments to consider that piles, when used, support the full load of the abutments and no allowance is made for any improvement in the bearing power of the soil.

Batter piles are always used by eight states and usually by six states. A number of states indicate that their unsatisfactory experience with abutments

pushing forward when founded on piles was due to the fact that batter piles were not used. This would seem to be a very important point. If batter piles are used with an abutment founded on piles, many states expect the forward movement to be negligible. As is to be expected, by far the largest percentage of piles are wooden, with only seven states reporting more than 50% concrete piles and one using more than 50% steel piles.

Ten states think that there is a definite correlation between the distance a ¾-in. round rod can be driven and the penetration of wood or concrete piles. Eight thought that there was no correlation. Eighteen did not answer, and the rest gave equivocal answers.

The *Engineering-News* pile formula is used more or less by forty-two states. Probably more use it, as a formula was mentioned by other states but not by name. In only one instance is a different standard named, and another state uses test loads exclusively. Usually steel piles are driven to refusal. Although the *Engineering-News* formula is almost universally used, there is a very widespread dissatisfaction with it, and a number of states ask for something better.

Under the heading, "What additional information should be brought out by this study?" came many interesting and pertinent suggestions covering moot points in the subject of abutment design. From these the Subcommittee condenses the following problems:

1. Develop a practical method for integrating the results of a soil analysis with design;
2. Determine the best practice in compacting the fill in back of the abutment in order to prevent bumps;
3. Determine what vertical load batter piles may be assumed to support;
4. Revise the *Engineering-News* pile formula or recommend another which will give more consistent results;
5. Determine whether abutments are being overdesigned; and
6. Study the effect of buoyancy.

Any one of these problems is a study in itself. There are committees of the various Divisions of the Society studying phases of several of these questions. To some of them there may be no definite answer; but in any case it is clear that the state bridge departments are very much alive to the importance of abutment design and construction. They are interested in obtaining more complete information concerning the foundation conditions and in improving their design methods and their construction technique, looking toward better and more economical structures. If some of the foregoing questions can be answered, it will be a step forward.

Respectfully submitted:

EDWARD A. MACLEAN

For Subcommittee of the Committee of the Structural Division on Masonry and Reinforced Concrete

APPENDIX

MASONRY COMMITTEE

OF

THE STRUCTURAL SECTION OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

A STUDY OF ABUTMENTS

IN

STATE HIGHWAY DEPARTMENTS

REPORT FROM........................STATE HIGHWAY COMMISSION

In the design and construction of abutments this Highway Department's procedure is as follows: (Explanatory remarks will be very useful in the cases where certain practices are followed because of local conditions or the availability of local materials)

Exploration.—The soil upon which the abutment rests is explored by: (Give usual method and supplement by comments on methods used in unusual structures)

- Sounding rod
- Auger
- Test pit
- Wash borings
- Loading platform
- Core drill
- Making a soil analysis

The soil explorations are made by:

- The regular survey parties
- Special survey parties
- The man in charge of the special survey parties is (is not) a specialist in the field of foundations
- His training is

The Fill.—The fill material used behind the abutment is:

- Ordinary fill, i.e., unclassified material used in approach fill.
- Gravel for........feet and to a depth of........feet or at a slope ....to........and a berm of........feet
- Material satisfactory to the engineer
- Material selected from a soil analysis
- Material with the optimum water content

The following special materials are used and precautions taken with a "buried" abutment:

....

....

The fill immediately behind the abutment is:

- Placed as approach fill (not) in layers
- The fill material is saturated with water

No water is allowed to be used

Special compacting devices are used such as:

. . . .

. . . .

Special care is used such as:

The following material is used in front of the abutment to prevent scour: Stone fill, steel sheet piling, or other methods.

Drainage.—The abutments are (are not) drained by:

Weep holes about . . . . feet apart and are above (below) the normal water line with the following method used to lead the water to them: Tile drains, stone drains, or gravel fill.

The following method is used to drain them:

. . . .

. . . .

Expansion Joints.—Expansion joints are (are not) required at about . . . . feet apart. At expansion joints the two sections of the abutments are (are not) keyed together and expansion material is (is not) used between the sections. The following method is used to keep the joint from showing unsightly leakages:

. . . .

. . . .

Reinforcing Steel.—Mass concrete abutments are (are not) reinforced:

In the footings

Where the wings join the breast wall

The entire exposed surface is reinforced with . . . . steel at . . . . inches on centers.

Design.—The abutment is analyzed by:

The Rankine theory of earth pressure

The Coulomb theory of earth pressure

Equivalent liquid pressure

Other theory

The earth is assumed to have an angle of repose of . . . . degrees, or an equivalent liquid pressure of . . . . lb per cu ft.

The angle of repose or equivalent liquid pressure is assumed to vary from job to job and is determined by: Judgment, or by a soil analysis.

Live load from the bridge is (is not) used as a superimposed load on the abutment.

Live load surcharge of . . . . lb per ft is used in back of the abutment.

A traction load, ... percentage of the live load, is carried by the abutment.

A back water pressure is (is not) assumed in front of the abutment.

A water pressure is (is not) assumed in back of the abutment.

If the buried, skeleton type of abutment is used is it designed or is it proportioned by judgment?

Outline the design procedure.

Types.—The following types of abutments are used with the estimated percentage of each type shown beside it:

Mass . . . .

Cantilever........
Counterfort........
Buried skeleton........
Semi-mass, i.e., gravity abutment with some reinforcement and pressure line outside middle third........
Others........

Architecture.—We have treated our abutments to improve their appearance as follows:

....

....

Plans and photographs are enclosed.

Unsatisfactory Experience.—Our experience has led us to fear chiefly the following methods of failure: Sliding, tipping, cracking, settlement, or washouts.

The causes of this unsatisfactory experience were:

....

....

Describe any noteworthy failures.
Pictures are enclosed.

....

....

The abutments are (are not) systematically inspected for cracks, scour, and other indications of failure every........years.

Piles.—Piles, when used, are considered to take the entire load of the abutment.

Piles are used to improve the bearing value of the soil.
In this case the abutments (do not) rest directly upon the piles.
A layer........feet thick of.................................... is used over the piles.
Of the piles we use........percent are bearing piles.
........percent are friction piles.
Of the piles we use........percent are wooden piles.
........percent are concrete piles.
........percent are steel piles.

Batter piles in front row and also in........other rows are:

Always used........
Sometimes used........
Never used........

An abutment on piles may be expected to move forward........inches.

A $\frac{3}{4}$-in. rod may (not) be driven as far as an ordinary wood or concrete pile can be driven.

The bearing value of piles is determined by:

....

....

What additional information should be brought out by this study?

Signed......................

Title........................

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

STRESS DISTRIBUTION AROUND A TUNNEL

Discussion

BY RAYMOND D. MINDLIN, ASSOC. M. AM. SOC. C. E.

RAYMOND D. MINDLIN,[33] ASSOC. M. AM. SOC. C. E. (by letter).[33a]—An analysis of the stresses around a horizontal tunnel of circular cross-section, situated in a gravitating elastic medium, is presented in the paper. It shows how the stresses are affected by (1) the proximity of the free upper surface of the medium, (2) a certain range of assumed initial stresses in the medium, and (3) variation of Poisson's ratio.

A result predicted from St. Venant's principle was verified—that is, when the tunnel is far from the surface the stresses may be calculated by considering the tunnel situated in a uniform stress field composed of the principal stresses initially present at the tunnel level. However, even in this comparatively simple case, there are available from the mathematical theory of elasticity only the solutions for openings of circular or elliptic section. Professor Richmond has shown how a very simple photo-elastic experiment, combined with a judicious use of elasticity theory, will solve the problem for any shape as long as the opening is far from the upper surface. In his application to the square section with circular fillets at the corners, Professor Richmond finds that there is an optimum fillet radius which produces a minimum stress concentration factor. This is a result of some importance which might be applied usefully in other fields of stress analysis and design.

Professor Richmond has listed correctly the limitations inherent in his suggestion. One of these in particular, which was also pointed out by Messrs. Brahtz and Feld, is of such importance in both the photo-elastic and mathematical treatments of the tunnel problem that it warrants another repetition. It is that the stresses in the vicinity of the tunnel cannot be predicted, even approximately, without advance information as to the state of initial stress in the medium. This was stated in the original paper when comparing the widely different results obtained for Cases I, II, and III.

NOTE.—This paper by Raymond D. Mindlin, Assoc. M. Am. Soc. C. E., was published in April, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1939, by Messrs. W. O. Richmond, and Jacob Feld; and February, 1940, by J. H. A. Brahtz, Esq.

[33] Instr., Dept. of Civ. Eng., Columbia Univ., New York, N. Y.

[33a] Received by the Secretary May 7, 1940.

Mr. Feld mentions the question of the effect of Poisson's ratio on the stress distribution. As stated in the paper, the reason for the great change in the stresses of Case II when Poisson's ratio is altered is that, in this case, the boundary conditions are functions of Poisson's ratio.

The effect on the stresses of bracing or lining in the tunnel, mentioned by Mr. Feld, can be calculated by the methods of elasticity theory. The effect of grouting pressure can be estimated from Jeffery's solution for the hole under uniform pressure in the semi-infinite plate. The results obtained from this solution are indicated in the first part of the discussion by Mr. Brahtz and are treated in greater detail in the source to which Mr. Brahtz refers.[29]

Mr. Brahtz reveals a very interesting point in his discussion of the problem of the semi-infinite plate with a circular hole near the straight boundary and under uniform stress parallel to the straight boundary. The point in question is whether or not there is a reversal of sign in the stress along the straight edge when the hole is close to this boundary. According to Jeffery's mathematical solution of the problem, there should be reversal of sign for $\frac{d}{r} < 2$. Mr. Brahtz made a photo-elastic experiment with $\frac{d}{r} = 1.5$ and found no reversal of sign, but he noted that the writer had called attention in his paper to an error in Jeffery's solution which prevents comparison with the experimental results.

The writer has corrected Jeffery's solution and finds that Mr. Brahtz's results are partly verified. The corrected solution indicates that there is no reversal of sign along the straight boundary. However, there are some discrepancies between the magnitudes of the stresses obtained in Mr. Brahtz's experiments and those found mathematically. These discrepancies result in percentage errors higher than the 5% estimated by Mr. Brahtz, especially in the regions of low stress. This is to be expected, since a small error in the measurement of phase retardation, while producing a negligible percentage error in regions of high stress, will produce a very large percentage error in regions of low stress.

The writer has calculated the corrected values for Jeffery's problem and has verified them photo-elastically for values of $\frac{d}{r}$ down to 1.1. The details of the results will be presented in another paper.

---

[29] *Technical Memorandum No. 597*, U. S. Bureau of Reclamation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

TENSION TESTS OF LARGE RIVETED JOINTS

Discussion

BY RAYMOND E. DAVIS, GLENN B. WOODRUFF, MEMBERS, AM. SOC. C. E., AND HARMER E. DAVIS, ASSOC. M. AM. SOC. C. E.

RAYMOND E. DAVIS,[49] GLENN B. WOODRUFF,[50] MEMBERS, AM. SOC. C. E., AND HARMER E. DAVIS,[51] ASSOC. M. AM. SOC. C. E. (by letter).[51a]—Such value as these tests may have in developing a concept of the action of riveted joints under steady loads has been enhanced by the interpretation of the results of these and other tests by the several discussers. The writers express their appreciation to those who have given their time to analyze the results and present their conclusions.

Considerable discussion has centered about the matter of design provisions for the calculation of net area in a riveted tension member. Former experimenters have noted that the customary omission of rivets at the edge of a connection has little effect on its strength, and no priority was claimed by the writers in pointing out that the effective net section of a member, as determined by experiment, does not agree with results obtained from formulas in common use. However, since designers have not generally recognized this fact in practice, the writers emphasized it by the proposal that riveted tension members be designed on the basis of their gross area. In making this proposal, the writers were aware that the data did not prove conclusively that such a procedure would represent the best possible practice. Additional analyses of the data of these and other tests, made by the various discussers, have confirmed the writers' belief that adoption of the proposal would make for an improvement over present practice.

It has been pointed out that the suggested unit stress of 16 000 lb per sq in. on the gross section for carbon steel is the equivalent of 21 300 lb per sq in. on the net section, and that such a unit stress is higher than permissible.

NOTE.—This paper by Raymond E. Davis and Glenn B. Woodruff, Members, Am. Soc. C. E., and Harmer E. Davis, Assoc. M. Am. Soc. C. E., was published in May, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1939, by Messrs. Charles F. Goodrich, Frederick P. Shearwood, and Jonathan Jones; October, 1939, by Messrs. C. C. Winter, W. M. Wilson, and J. M. Garrelts; April, 1940, by A. E. Richard De Jonge, M. Am. Soc. C. E.; and May, 1940, by E. C. Hartmann and Marshall Holt, Assoc. Members, Am. Soc. C. E.

[49] Prof., Civ. Eng., Director, Eng. Materials Laboratory, Univ. of California, Berkeley, Calif.

[50] Cons. Civ. Engr., San Francisco, Calif.

[51] Associate Prof., Civ. Eng., Univ. of California, Berkeley, Calif.

[51a] Received by the Secretary May 6, 1940.

Table 26 of Mr. Winter's discussion shows the close relation between the stress computed on the basis of the least net area and the strength of mill coupons. Such studies show that the unit stresses proposed are approximately those that obtain under the present practice of designing for 18 000 lb per sq in. on net section and taking the usual care to protect net section. Mr. Goodrich asks that designers be given a chance to use their ingenuity in protecting the net section. The writers believe that these attempts at ingenuity generally result in a design that has less efficiency than one in which the attempt to protect net section is disregarded. Mr. Goodrich mentions the case of a single hole out of an angle. Although no experimental data covering this case are known to the writers, they doubt that the ultimate strength, based on gross section, would materially exceed 75% of the coupon strength.

Professor Wilson's proposed formula corresponds very well with the available experimental data. It allows for the grooving effect with close pitches of rivets, and it properly penalizes wide pitches, although from the standpoint of practical design the formula would again tempt the ingenuity of the designer. The writers find no reason to doubt that full end riveting of the splice plates with a $\frac{G}{D}$-ratio of 4 represents as good practice as can be developed from existing information. Professor Wilson finds that this arrangement gives an efficiency of 80 per cent. To be on the conservative side the authors suggested a ratio of net to gross section of 75 per cent.

Mr. Jones' suggestion of a weld at the end of splice plates introduces a field for further experiments, and further knowledge as to the weldability of various steels may lead to economies in design. Naturally such a procedure would be limited to steels suitable for welding and might require an alloy other than silicon for tension members. Even with supplementary welding, however, the assumption should not be made (until after its truth is developed by experiment) that the strength of the splice will not depend upon the least net section.

It appears to be the consensus of opinion that friction between the plates cannot be depended upon to any material extent, and that the holes should be filled as completely as possible. Mr. Goodrich feels that the optimum with respect to filling of holes by rivets has been reached. Possibly specifications for steel structures should fix tolerances for the diameter of rivets and for the size of the holes. In the case of the San Francisco-Oakland Bay Bridge both the quality of fabrication and the standards of inspection were above the usual commercial standards. The measurement of rivets and holes, by calipers, developed the fact that the rivets were generally undersized and the holes generally oversized. Field tests of driving rivets developed that, with the field connections reamed, there was no trouble in entering the rivets when the diameter of the cold rivet was $\frac{1}{16}$ in. less than the diameter of the hole.

Mr. Goodrich disagrees with Recommendation (6) in regard to the use of manganese-steel rivets. The writers have little doubt that, in any actual design, Mr. Goodrich would agree with the writers as to joints, including gusset-plate connections, in which manganese-steel rivets should be used. The writers feel that, in general, present design practice tends toward the extreme in attempts to reduce weight of steel in structures, and that alloy structural steel

or rivets should be used only when the economies gained by using the higher-strength materials are distinctly worth while.

The writers believe that Mr. Shearwood's criticism of the description of the action of riveted joints arose from his failure to realize that the action as shown in Fig. 1(*b*) was based on a simple specimen with only two rows of rivets. The action shown by Fig. 1(*d*) is representative of the cases illustrated by Mr. Shearwood. In this latter case, end slip occurs at very low average intensities of stress on the rivets; middle slip does not start until after considerable slip has occurred near the end of the joint.

Mr. Shearwood's suggestions as to cold-driven rivets open another field for further development. At present, the equipment for driving them is so heavy that they cannot be considered for field work.

Mr. Jones suggests that the ultimate capacity of a joint will suffice as a basis for selection of design stresses. This suggestion should be applied with caution to the results of tests on large joints. As an example of data which might lead to false conclusions, the behavior of the heavier specimens of the F series may be pointed out. These specimens were designed to fail in the plates. The first failures took place in the rivets, and occurred because of yield in the plates sufficient to shear the end rivets. It is possible that if the number of rivets in these specimens had been doubled, the rivets might have failed at the same total load, and the conclusion might have been drawn that the rivets had only half the unit strength actually developed.

Mr. de Jonge's discussion is valuable in suggesting fields for further research. He has also added an interesting explanation of the failure of wide plates without rivet holes. The writers believe that a careful reading of the paper will clear up many of the difficulties expressed by Mr. de Jonge. Space limitations called for extreme brevity in view of the amount of original data presented, so that neither speculative interpretation nor presentation of extensive detailed data was possible.

A few of his comments appear to require answer. The writers are unable to agree with his main thesis in regard to the development of a "rational theory of riveted joints." Five conditions of perfect fabrication were given in the paper. To these might be added: Uniform tension in all rivets, approximating the yield strength of the rivet material. With the most perfect practicable fabrication, these conditions cannot be attained. For example, tension in the rivets of a joint may vary from zero to the yield strength of the material. Furthermore, there can be no doubt that considerable plastic action takes place at comparatively low loads, thus invalidating the assumption of elastic rivet and plate materials, on which the proposed "rational theory" of necessity would be based. Any theorizing, therefore, must be based on assumptions which cannot be met in practice.

The discussion of rivet temperatures introduces the question of greatest possible tension in the rivets *versus* completeness of filling the holes. Possibly the decision should be on the basis of type of service required of the joint. For ship plating, or for members subjected to reversals of stress, high rivet tensions may be desirable. In this case it may be that the rivet temperatures suggested by Mr. de Jonge, and the "smallest possible pressure of the ram," should be

used. However, for members subjected to comparatively steady stress in one direction, the evidence is clear that slip will occur within the range of working loads. In this case it appears desirable that the holes be filled as completely as possible. To achieve this result it appears necessary to heat the rivets to as high a temperature as possible without burning them, and to use such driving pressures as to upset the shank of the rivet completely. In either case, it is desirable that the bolting up prior to riveting bring the faying surfaces into contact, and that the action of the riveter be continued longer than is usual.

In view of the foregoing considerations, the writers are unable to agree with Mr. de Jonge's Recommendations 1, 2, and 5.

The writers agree with Mr. de Jonge as to the importance of understanding the action of riveted joints within the yield range, and it was with this idea in mind that so many strain measurements were made. However, as pointed out by Mr. Jones, the indications of these tests are that the ratio of rivet yield to plate yield does not differ greatly from the corresponding ratio at the ultimate. From these and other considerations, the writers cannot agree with Mr. de Jonge's Recommendation 6.

There appears to be no reason to rule out shingle-type joints, as Mr. de Jonge does in his Recommendation 7. This type of joint is often more economical of material than a corresponding straight butt joint, and is often necessary as a means of keeping the rivet grip within reasonable limits.

It is believed that Mr. de Jonge has misapplied the idea of the "true stress-strain diagram" calculated from the test of a steel bar in tension to the detrusion failure of rivets. In the failure of structural parts or members, one is concerned with the actual load (or total load *versus* total deformation) required to cause failure, and not with a unit stress calculated by dividing the load by reduced cross-sectional area.

On the matter of joint elongation, it was the writers' intention to show that for calculations involving elongation of members such as are necessary in computing deflections, in computing secondary stresses, or in analysis of statically indeterminate structure, it is unnecessary to take the joints into separate consideration. A joint occupies only a short proportion of the length of a member. The extreme ratio of 3.32 was for a joint which probably would not be used in practice.

Messrs. Hartman and Holt have contributed valuable additional data on the problem of the effective net section; the information having to do with aluminum alloys, small rivets, and very thin plates is of particular interest.

It is pointed out by Messrs. Hartman and Holt that the use of the A.R.E.A. formula for effective net section could be applied to the results of the University of California tests to compute the ultimate loads within an average error of about 10% (range, 30%). Although this may be true, the effective net widths computed by means of this formula bore no relation to the results of tests. Close agreement was found between values of effective net width, and width determined simply on the basis of full net section, without the use of any formula. In this connection, it may be well to emphasize that the tests reported in the paper were concerned with riveted connections having a number

of rows of rivets such as are used in bridge construction, rather than with connections with few rows of rivets such as are used in ship and tank construction.

Mr. Jones' suggestion that further experimentation may well be done on small specimens is well taken. As expressed in the "Introduction" to the paper, the reason for the large specimens was that those responsible for the tests felt that current practice represented too extended an extrapolation of previously existing experimental data which had been obtained from tests of small specimens. As mentioned by most of the discussers, the confirmation of major design practices was a valuable result of the investigation.

It appears that all research programs develop the need of additional ones. From the standpoint of contribution to the art of structural design, the following appear to be most worthy of additional study at the present time:

1. Further confirmation of the conclusion that greatest actual efficiency is obtained by full-row riveting;

2. Determination of the ratio between gage and rivet diameter, for most economical results; and

3. Investigation of Mr. Jones' suggestion with regard to the use of a weld at the end of splice or gusset plates.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

THE UNIT HYDROGRAPH PRINCIPLE APPLIED TO SMALL WATER-SHEDS

Discussion

By E. F. Brater, Jun. Am. Soc. C. E.

E. F. Brater,[20] Jun. Am. Soc. C. E. (by letter).[20a]—The writer is gratified by the stimulating comments contributed by the various discussers. Some of the discussions provide additional valuable data. Several points of controversial nature were raised and discussed, and the writer wishes to add his views on these several points. In general, it should be said that the analysis was based on a study of twenty-two small southern Appalachian water-sheds. The results reflect the influence of the topography, soil, and vegetation of this region. It must be recognized, therefore, that some of the conclusions may have to be modified if they are to be applied to other localities.

The main purpose of the paper was to study the application of the unit hydrograph to small water-sheds. It seemed desirable to keep the discussion, as far as possible, in terms of well-known and accurately definable quantities. For this reason, run-off coefficients were used instead of the newer conception of infiltration capacity. The run-off coefficient is simply the percentage of a given rain which appears as flood flow in the stream. Its meaning is clear and straightforward and its value may be determined easily, within reasonable limits of accuracy.

Mr. Sherman suggests, however, that the paper would have been more valuable if infiltration capacities had been used. He has computed the infiltration capacity by the direct method for a number of the hydrographs given in the paper and has presented the results along with values from other parts of the United States.

The writer does not question that the infiltration capacity as defined by Robert E. Horton,[7] M. Am. Soc. C. E.—namely, "The maximum rate at which

Note.—This paper by E. F. Brater, Jun. Am. Soc. C. E., was published in September, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Franklin F. Snyder, Jun. Am. Soc. C. E.; February, 1940, by Messrs. LeRoy K. Sherman and Raphael G. Kazmann; and April, 1940, by Waldo E. Smith, M. Am. Soc. C. E.

[20] Instr., Civ. Eng. Dept., Univ. of Michigan, Ann Arbor, Mich.

[20a] Received by the Secretary April 30, 1940.

[7] "Surface Run-off Phenomena, Part I—Analysis of the Hydrograph," by Robert E. Horton, *Publication 101*, Horton Hydrological Lab., Voorheesville, N. Y.

a soil, when in a given condition, can absorb moisture"—is a meaningful and important soil characteristic. It should be recognized, however, that the so-called infiltration capacity computed by the direct method is not the quantity defined by Mr. Horton and does not have any real physical significance. This may be rather forcefully illustrated by considering the three values which Mr. Sherman has obtained for Coweeta 7. It will be shown that there was no overland flow during these three rains, and therefore that the actual infiltration capacity was not reached.

The length of stream of Coweeta 7 is 11 100 ft. The width of the flowing stream during a heavy rain varies from about 20 ft to a mere trickle, but the gravelly bed of the stream plus the adjacent banks embraces a strip 20 to 30 ft wide over the length of the stream. Therefore, if all of the flood flow from any given rain can be accounted for by 100% run-off from a strip of land 11 100 ft long and not in excess of 25 to 30 ft wide, it must be concluded that no overland flow occurred. A rainfall on this water-shed of 0.92 in. on June 12, 1936, produced a total flood flow of 6 850 cu ft. This is equal to a volume of water 0.92 in. deep on an area 8.1 ft wide and 11 100 ft long. An 8.1-ft strip barely accounts for the actual area of the flowing stream, so there could not possibly have been any overland flow.

Similarly, rains of 3.98 in. on April 1, 1936, and 5.95 in. on September 29, 1936, produced volumes of flood flow of 50 000 cu ft and 64 500 cu ft, respectively. The widths of 11 100-ft strips from which 100% run-off would have to occur to produce these flood flows are 13.6 and 11.7 ft for these two storms. Here again there was no overland flow.

Since no overland flow occurred, the infiltration rates were simply equal to the rainfall rates, and the infiltration capacity, as defined by Mr. Horton, was not reached; yet Mr. Sherman has computed values of infiltration capacity of 2.35, 2.76, and 2.10 in. per hr, respectively, for these three rains. It is believed that the actual infiltration capacity of Coweeta 7 is more than 5 in. per hr. Overland flow has never been observed from any of the heavily forested areas in this region, except during severe cloudbursts.

It would be highly desirable to be able to compute the actual average infiltration capacity of a water-shed from the rainfall and run-off records. One of the requirements necessary to approach this ideal more closely would be to take into account the part of the water-shed from which 100% run-off occurs. This would include the areas of the lakes and streams as well as all impervious portions of the water-shed. The result of neglecting this factor is illustrated by the foregoing examples. Because of its variable nature, however, it would be difficult to estimate, accurately, the area of 100% run-off. For instance, during a prolonged, heavy rain, not only does the area of the permanent lakes and streams increase, but this area is frequently augmented by additional inundated areas. The error introduced by neglecting impervious areas causes computed values to be too small.

It should also be pointed out that even if the impervious areas and the areas of lakes and streams were carefully taken into account, the infiltration capacity computed by the direct method might still differ considerably from the actual infiltration capacity. This method of computation is based on the

assumption that the rainfall excess is equal to the surface run-off. For this assumption to be true, it would be necessary for all the surface detention present at the end of each period of rainfall excess to be transported instantaneously into the flowing stream. This is not true, of course, since most of this surface detention has an additional opportunity of infiltering for periods varying from a few minutes to several hours. Therefore, an error is introduced, the magnitude of which depends on the variation in rainfall intensity. Another error which is inherent in this method of computing infiltration capacity results from the fact that depression storage is included with the total infiltration. All water left standing at the end of rainfall excess, whether it be in tiny puddles or good-sized lakes, is slowly dissipated as evaporation or infiltration. This process may continue for a number of days, and yet the direct method assumes all of this water to have been infiltered during the period of rainfall excess. This factor has little or no effect in the case of the water-sheds discussed in the writer's paper, but may be very important in flat or terraced water-sheds. The errors discussed in this paragraph are compensated for, to some extent, in the case of relatively impervious water-sheds, by residual, low-intensity rainfall which may follow the period of rainfall excess.

The values computed by this method may be very useful quantities for some purposes. It seems highly desirable, however, to avoid calling them infiltration capacities since that implies that they represent the maximum rates at which a given water-shed can absorb rainfall. For instance, they could be called infiltration indexes or rainfall excess indexes.

Mr. Snyder and Mr. Smith ask whether the values of duration of rainfall are corrected for initial lag. The initial lag is small on these streams, since the stream rise begins almost at the beginning of rainfall. The values given are corrected for any minor, low-intensity rainfall that may occur at the beginning and end of the storms. The period given is very nearly equal to the duration of run-off producing rain.

Mr. Sherman points out that the shape of the distribution graph is influenced by the rainfall pattern. Mr. Snyder and Mr. Smith note that the period of rise is not a constant quantity. These are pertinent observations because they emphasize the fact that the unit hydrograph is not a mathematically precise method for predicting the nature of flood flow but simply a workable one, sufficiently accurate for all practical purposes. The original study consisted of an analysis of seventy-eight storms on twenty-two water-sheds. The mean of the individual variations from the average period of rise on all of the water-sheds was about 25 per cent. Part of this variation is due to the presence or absence of a small fillet preceding the main portion of the rising hydrograph. The general shape of the hydrograph is not materially affected thereby.

Mr. Snyder concludes that this variation in the period of rise is due to the variation in the duration of rainfall. The writer indicated in the paper that the period of rise tends to remain constant despite variations in rainfall duration, provided the duration does not greatly exceed the period of rise. Another careful comparison, by the writer, of the period of rise with rainfall duration on all twenty-two of the water-sheds shows no evidence of a direct relationship

between the two quantities. Mr. Snyder rightfully cites as evidence of the effect of rainfall duration on the period of rise the statements made by the writer concerning the method of breaking down continuous rainfall records into individual storms for use in reconstructing hydrographs from rainfall data. The paragraph to which Mr. Snyder refers reads in part as follows (see heading "Applications of the Distribution Graph: Application of the Pluviagraph"):

> "In general, the rain was divided according to changes in intensity. However, if any rate of rainfall continued for too great a time, it was found necessary to break it up arbitrarily into parts, especially if the rain was of low intensity. In the case of Copper Basin 2, and Coweeta 7, a half-hour was found to be an adequate unit of time into which to break these rains."

The writer did not, however, intend to imply that periods greater than 30 min would not produce adequate results because periods as long as 130 min were actually used.

Table 8 shows the manner in which the continuous rainfall records were

TABLE 8.—Manner in Which Two Continuous Rains Were Broken Down for the Application of the Distribution Graph

(a) Storm of April 1 and 2, 1936 (Intervals from 9:30 a.m. to 5:00 a.m.)						(b) Storm of September 29 and 30, 1936 (Intervals from 5:15 p.m. to 4:15 a.m.)		
Time at beginning and end (1)	Duration, in minutes (2)	Precipitation, in inches (3)	Time at beginning and end (1)	Duration, in minutes (2)	Precipitation, in inches (3)	Time at beginning and end (1)	Duration, in minutes (2)	Precipitation, in inches (3)
9:30– 9:45	15	0.06	10:00–11:00	60	0.12	5:15– 6:45	90	0.32
9:45–10:15	30	0.14	11:00–11:45	45	0.04	6:45– 7:45	60	0.20
10:15–11:00	45	0.06	11:45–12:15	30	0.23	7:45– 9:00	75	0.20
11:00–12:45	105	0.06	12:15– 1:00	45	0.82	9:00–10:00	60	1.55
12:45– 2:30	105	0.18	1:00– 1:15	15	0.06	10:00–11:15	75	0.58
2:30– 3:00	30	0.05	1:15– 1:30	15	0.24	11:15–11:30	15	0.32
3:00– 4:00	60	0.06	1:30– 2:15	45	0.12	11:30–12:00	30	1.22
4:00– 4:45	45	0.08	2:15– 2:30	15	0.35	12:00– 1:00	60	0.14
4:45– 5:15	30	0.08	2:30– 3:15	45	0.06	1:00– 2:00	60	0.14
5:15– 5:45	30	0.08	3:15– 3:45	30	0.17	2:00– 2:45	45	0.47
5:45– 6:30	45	0.18	3:45– 4:30	45	0.17	2:45– 3:30	45	0.22
6:30– 7:00	30	0.09	4:30– 5:00	30	0.06	3:30– 4:15	45	0.25
7:00– 7:45	45	0.24	5:00–	....	0.08	4:15–	....	0.32
7:45–10:00	135	0.11	....	....	....	....	....	....

broken down in order to reconstruct the hydrographs presented in Fig. 8. Although it may not be completely clear, due to the small scale of Fig. 8, the intervals chosen correspond to changes in rainfall intensity. The operation was performed without giving any attention to the period of rise. As a result, there is no regularity in the length of time intervals. It may be seen from Table 8 that the intervals vary from 15 min to 135 min. Since the actual hydrographs are quite faithfully reproduced by the reconstructed ones, despite this variation in the length of interval, it appears that the duration of rainfall has only a minor effect on the period of rise. There are other factors which cause the length of the period of rise to vary. The influence of the rainfall pattern was mentioned by Mr. Sherman. Other factors of importance in these

small, steep water-sheds are the intensity at which rainfall begins, and the direction of approach of the rainstorm.

Mr. Smith isolates the following phrases as evidence of the relationship between duration of rainfall, within the period of rise, and the length of the period of rise: "that a 30-min interval was too long * * * whereas a 15-min interval produced satisfactory results." It is only necessary to read, in its entirety, the paragraph from which this quotation was taken to realize that the time interval referred to is simply the result of the number of divisions into which the unit hydrograph was separated, arbitrarily, in order to compute the distribution graph percentages. In reconstructing a hydrograph from rainfall data, these percentages are applied to the total rainfall to determine the portion contributed to stream flow in each corresponding interval. These portions are correctly plotted as blocks having a width equal to the time interval and height equal to the average rate of flow necessary to produce the given quantity of run-off. In order to construct a smooth curve through the tops of these blocks, they must be made reasonably numerous, especially when two or more of such unit hydrographs are superimposed upon one another. There can be no connection between this purely mechanical or drafting-board procedure and the period of rise of the unit hydrograph.

Mr. Smith makes the following statement in his discussion: "If rainfall of uniform intensity in excess of the infiltration rates continues for the duration of the distribution graph, the period of rise will be found to be about equal to the duration or base of the distribution graph." This statement is not consistent with the meaning of the period of rise as used by the writer. The hydrograph described by Mr. Smith is the combination of two or more unit hydrographs. Therefore, the rising part of the hydrograph consists of portions of two or more periods of rise, and should not be referred to as the period of rise.

Mr. Kazmann states that a decrease in run-off coefficient in the second of two consecutive rains on Bent Creek 3 might have been due to variations in rainfall intensity. The mass diagrams of rainfall for the two rains are quite similar; therefore, it is believed that the dry period of 1 hr which separated the rains permitted the soil to regain some of its capacity for the absorption of water.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

COMBINING GEODETIC SURVEY METHODS WITH CADASTRAL SURVEYS

Discussion

By Carl M. Berry, Assoc. M. Am. Soc. C. E.

Carl M. Berry,[12] Assoc. M. Am. Soc. C. E. (by letter).[12a]—There is little question but that land-acquisition practise making use of the State-wide conformal systems, with these in turn dominated by geodetic control, is steadily establishing itself throughout the membership of the engineering fraternity. Indeed, the improvements to follow in title-transfer practise may well be stated as the logical corollaries of a controlled boundary survey, the adoption of which among surveyors, title examiners, and the inestimable host of the laity who are to some degree affected by land titles and boundaries of real property will manifest itself in the future as progress—a progress which will depend, as it has in the past, upon growth in understanding.

The comment of Professor Sawyer, wherein he "doubts the advisability of attempting second-order and third-order triangulations by the program outlined in the paper, except for those surveying organizations best equipped in personnel and instruments," is well taken. Although overemphasis of this point might, in effect, lend hesitancy to those organizations contemplating the use of control surveys, it is felt, nevertheless, that such retarded impetus would be preferable to the questionable results of ill-planned and executed projects which might otherwise result. The long-range objectives involved in the use of modern technique should not lose sight of the public-relations aspect of the problem.

Professor Kissam's discussion might well be augmented by the appeal in his excellent paper,[8] "Proposed Improvements for Land Surveys and Title Transfers," for surveyor and title examiner to combine in a joint effort to find ways and means for employing those excellent tools already at hand (modern survey equipment, skill, procedure, and established State systems of plane

Note.—This paper by Carl M. Berry, Assoc. M. Am. Soc. C. E., was published in September, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1939, by William L. Sawyer, Assoc. M. Am. Soc. C. E.; January, 1940, by Philip Kissam, Assoc. M. Am. Soc. C. E.; and February, 1940, by Gordon Macleish, Assoc. M. Am. Soc. C. E.

[12] Civ. Engr., U. S. Bonneville Power Administration, Portland, Ore.

[12a] Received by the Secretary May 8, 1940.

[8] "Proposed Improvements for Land Surveys and Title Transfers," by Philip Kissam, *Proceedings*, Am. Soc. C. E., April, 1939, p. 573.

co-ordinates) toward eliminating boundary difficulties. His suggestion of the use of a more modern type of direction instrument is one to be considered seriously by any organization contemplating work of this nature. The writer fully agrees that the advantages pointed out by Professor Kissam would result. His further comment on the use of a note to indicate use of "grid" bearings on the right-of-way parcel plat is believed covered by the note, "Grid Bearings, Distances and Coordinates Are Based on the North Washington System of Plane Coordinates," which obviously applies to the tabulated values in the box "Reservoir Boundary," as well as to the values along section and subdivisional lines on the upper part of the sheet. Perhaps a clearer expression would result by substitution of the word "All" for "Grid" in the note, inasmuch as all values shown on the plat are in terms of "grid" values; the theta mapping angle is represented principally for the convenience of surveyors and title examiners in plotting previous deed calls by metes and bounds which would ordinarily follow "true" bearings. Plats of this type have met wide-spread favor among abstract and title companies, because of their scale representation of parcels and ownerships, and were generally used, in lieu of the usual lengthy and cumbersome typewritten metes and bounds descriptions, in negotiating the purchase contracts from landowners.

The writer has been identified with the U. S. Bonneville Power Administration engaged in right-of-way acquisition for transmission lines in the States of Oregon and Washington, and has developed a plat embracing a square mile —one plat for each section of land invaded. Through use of a single master tracing for each section, the individual ownerships, often numbering twenty or more to the plat, are represented by coloring each parcel, a device saving much drafting room time and labor. At the same time much of the attendant possibility of errors in the preparation of the maps is removed. Although neither of these two States has yet adopted the Enabling Act, nevertheless it is felt that much groundwork has been laid toward its early introduction and passage.

The computing methods suggested by Mr. Macleish indeed look to be engagingly neat, concise, and easy to perform; certainly the forms require a minimum of written figures. As with all new designs, however, the writer would want to see an average group of computers utilize these forms for a month or so before concluding that the Macleish forms were definitely superior, the principal points of skepticism being (1) the recommendation to perform a maximum number of operations on the machine without jotting down intermediate figures, and (2) the necessity for remembering "rules-of-thumb" regarding signs of angles and choice of alternative "*a*" and "*b*" routes in the computations. Of course, most persons of ordinary ability and interest would have no trouble remembering what criteria governed in the different stages of the computations, but, as is frequently the case, not all members of the computing organization are of that caliber and for this reason it follows that whatever forms are used should be freely labeled to be as self-explanatory as possible. Forms should be designed so that the casual or occasional computer would not become lost in endeavoring to carry out intricate or involved computations on the keyboard of the machine—the steady or experienced computer will involun-

tarily take the short cuts anyway, whether the form calls for intermediate results or not. One of the most noticeable traits of computers is "going stale" on the computing forms after having been diverted to other drafting or computing work for occasional periods of a few weeks or so. In such cases the presence of ample labels and "mile-posts" in the computing form is most helpful.

From an administrative point of view, recognition must be given the fact that the majority of computers are recruited either from lists of applicants, often scantily qualified in both mathematics and calculating machine operation, or from chainman-rodman ranks during periods of inclement weather. Thus, computing forms should possess an essence of simplicity conducive to short training periods, and at the same time offer the necessary means of checking, mechanically, as the solution is being "cranked through." One of the worthy features of the Bureau of Reclamation three-point solution is its elimination of an inverse solution between the main scheme stations; often these values are not available from the geodetic reduction, particularly in such cases as water towers, spires, and other points originally located by second-order as intersection stations. Although the possible use of the check-position cut was mentioned in the discussion of the three-point solution, and further laudably mentioned in the discusser's conclusions, the Macleish forms have not provided for incorporating the "check-position" cut in each solution—a feature which the writer emphasizes as of utmost importance in any cadastral surveying technique which departs from the conventional ground methods.

Aside from the matter of computing forms, Mr. Macleish questions the justification for using tenths of seconds. The writer's decision to use tenths of seconds throughout was made, not as an attempt to imply a higher degree of accuracy than was actually attained, but rather to unify all computation work by removing from the computer's mind unnecessary mental "traffic signals" through the several steps of the computation. What actual justification there might be for this practise would doubtless be limited to the following:

On long lines of observations and weak fixes or intersections, the field-observed directions might actually be accurate within 0.5 sec. In such cases it seems quite worth while to compute the point with the use of tenths of seconds rather than round off the directions to the nearest whole second and risk distorting an angle from 0.5 to, say, 1.5 sec. If, then, it is worth while to compute an occasional point with tenths of seconds, it is logically economical to compute all third-order work with tenths of seconds, for unless one is using tables of functions listed for every whole second, the extra time required to interpolate the function of tenths of seconds on the machine is negligibly greater than the time required to interpolate to whole seconds. Moreover, computers work most efficiently if they handle the same number of decimal places in seconds, distances, etc., in all the operations throughout the day. For example, in occasionally carrying computations to tenths of a foot instead of to hundredths of a foot it was disclosed that in the long run there was more time lost than gained, because the computer had to stop to think that he must not put down the hundredths in his computations.

Mr. Macleish has commented regarding the requisite of a ten-bank computing machine for the Bureau of Reclamation intersection computation form. Although this was never a matter of concern inasmuch as it seems to be the policy of the Bureau to purchase only ten-bank machines, the eight-bank machine could nevertheless have been used in all cases with but little inconvenience. To use an eight-bank machine the first two figures of the X co-ordinate would be omitted in forming Equation (1). After adding the two equations the X-value thus obtained by dividing would then be, say, 55 948.40 to which would be added the figures previously dropped to obtain the true X co-ordinate, 2 655 948.40. The large cotangents of lines bearing approximately north and south are no trouble because the rate of change of X is so small. On these very nearly north-south lines one can almost estimate the X co-ordinate before looking up the cotangent; to obtain the true X-value of the station the computer simply feeds the machine the customary number of significant figures, nine or ten if using a ten-bank machine, seven or eight if using an eight-bank machine, and the customary accuracy results in the answer. The only hazard of large cotangents is that an inexperienced computer may compute the Y-value of the intersection station by substituting the X-value in the equation having the large cotangent. However, if he immediately checks his Y (as he always should) by substituting X in the other of the pair of equations, he finds the Y-value does not check very closely and thereafter knows better than to expect north-south lines to offer accurate Y-values. Mr. Macleish has spent much careful thought and analysis on the subject, and is to be complimented for the masterly manner in which he investigated the subject and presented his discussion.

It is indeed gratifying that the discussers are all engineers who are well acquainted with this field of work, and the writer feels that an impressive assurance has been given a method which has as its objective an economical procedure for a controlled, permanently marked, and recorded land-acquisition survey.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DEVELOPMENT OF THE COLORADO RIVER IN THE UPPER BASIN

Discussion

BY THOMAS C. ADAMS, M. AM. SOC. C. E.

THOMAS C. ADAMS,[7] M. AM. SOC. C. E. (by letter).[7a]—Mr. Elder's discussion is directed mainly at two points. One is the accuracy and adequacy of the writer's analysis of the division of water supply between the Upper and Lower Basins and the apparent intended use of water in the Lower Basin. The other is the justification of Government subsidy for water-supply development.

Consider first the matters of water supply. Table 2 is an application of the water-supply partition provisions in the Colorado River Compact to the distribution of the water of the river (and purports to be nothing else). The writer was led to prepare it because one of the problems vital to Upper Basin interests is the quantity of water available to the Upper Basin under the compact provisions and in conformity with the actual flow of the river. Table 1, giving estimates of water likely to be demanded by Lower Basin projects, was prepared because some believe that aggressive advancement of irrigation and other water-supply construction in the Lower Basin, with consequent large demand for water, is likely to result in an encroachment upon water supply generally supposed to be reserved by the Colorado River Compact for the use of the Upper Basin. In so far as the Upper Basin is concerned, the Colorado River Compact had an outstanding purpose—namely, to reserve an equitable portion of the undeveloped supply in the river system so that the Upper Basin might find it available for use at any future time. Furthermore, it is now coming to be more firmly believed that if projects of the Lower Basin make use of quantities of water so large as to encroach on the water supply reserved by the Compact to the Upper Basin, this water cannot be recaptured for Upper Basin use even though the compact provisions are explicit. One factor supporting this view is that it would immediately appear uneconomical to restrict the use of water by an operating project which involves a large investment, works of monumental proportions, many human settlements spread over a vast area of

NOTE.—This paper by Thomas C. Adams, M. Am. Soc. C. E., was published in September, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1940, by C. C. Elder, Assoc. M. Am. Soc. C. E.

[7] Associate Prof., Civ. Eng., Univ. of Utah, Salt Lake City, Utah.

[7a] Received by the Secretary May 7, 1940.

land, and far reaching, established, economic relations, in order to permit this water to be used on other, undeveloped lands where works are to be built, and all economic and other social advances are to be made.

Within the ten years 1930–1940, large developments in both the Upper and Lower Basins of the Colorado River have been, and in the future most likely will be, involved with Government financing dependent upon Congressional approval (and as some view it, also involved with a measure, greater or less, of Government subsidy). Therefore, the possibility of developments involving the withdrawal of water from use in one locality for use in another, even in conformity with previous agreements and with acknowledged equities, appears even more unlikely. An important factor in such a situation is that the wrong that might be done is not to an individual but is a more or less intangible wrong done to a community or section of the country, a situation which does not elicit so strong a personal interest from those who might defend equities nor so strong a public opinion upholding action with the same intention.

All should take special note that the question of the desirability of a legal formula which would withhold a natural resource for development in a specific locality at a late date is not now under discussion. In so far as this applies to the water of the Colorado River System, it was decided by contract when the Colorado River Compact was drawn and ratified.

It will bear re-statement that Table 2 was prepared to show the application of the partition terms of the Colorado River Compact to a period of river flow actually occurring. The partition terms apply to the reconstructed flow that was presumed to take place under pristine conditions. The terms are comparatively simple, and the provision is one of the most important in the Compact. Complications arise mainly from vagaries of river flow and the minds of men. Both of these are very complicated indeed; but very few have been bold enough to state that the Compact provisions will be set aside.

In the preparation of Table 2 it was necessary, of course, to use some judgment in the selection and ordering of data. In the few instances, where choice was not precluded by the definiteness of the data and strictly mathematical nature of operations, the writer chose to give preference to data and interpretations from authoritative sources. Such is the case with values in Columns (4) and (5) which record the Upper Basin annual demands for water under conditions of actual development in corresponding years, and future intended development for conditions of these years under terms of the Colorado River Compact, respectively. It is to be noted that later in Column (12), Table 2, it is found that deductions from the latter values—those of Column (5)—for years after 1936 are necessary because of reduced water supply.

The values of Column (5), Table 2, are not a constant 7.5 million acre-ft per yr because, in years of high runoff, water users will be able, and will find it convenient, to divert somewhat more than they normally divert. Where diversion is made by many thousands of small ditches (a condition that exists in the Upper Basin), it would be a great feat of water administration—one that has not been generally accomplished—to correct this condition. In years of low runoff, conversely, diversions will be below average for similar reasons; but the departures of values in Column (5) from 7.5 million acre-ft are not great. The largest of the values, 8.8 for 1929—a year of rather great stream flow—

should compare with 12.5 million acre-ft from direct river flow available to the Lower Basin during this same year, not with 6.2 million acre-ft as inferred by Mr. Elder. However, if a uniform 7.5 million acre-ft per yr had been assumed in Column (5), the important conclusions drawn from Table 2 would not have been changed—namely, that (1) the water supply is critically limited; (2) the water partition provisions of the Colorado River Compact, although satisfactory when the river flow is high or moderate, have peculiar, undesirable consequences on the distribution of water in the presence of intended (future, ultimate) Upper Basin use of water and when prolonged, but to be expected, droughts occur.

Conclusions originally made from Table 2 are emphasized when this table is extended using discharges actually recorded in 1938 and 1939 in place of the imaginary values at first used. These discharges have become available since the table was prepared. The actual discharge for 1939 was considerably less than the imaginary value first used. An addendum for Table 2 is given in Table 4 using the actual values of discharge for 1938 and 1939 and an imaginary

TABLE 4.—Discharge of the Colorado River at Lee's Ferry, Presenting Distribution of Water Between Upper and Lower Basins as Required by the Colorado River Compact

(Addendum to Table 2; All Values in Millions of Acre-Feet per Year)

Year	Measured discharge, in millions of acre-feet per year	Ten-year cumulative discharges	Estimated Annual Depletion		Columns (2) and (4), minus Column (5)	Ten-year cumulative values from Column (6)	Measured discharge at Bright Angel, Ariz.	Ten-year cumulative values from Column (8)	Column (8) minus Column (2)	Annual estimated deficiencies	Water available for consumptive use	Annual discharge at Lee's Ferry based on conditions affecting Column (12)	Annual water supply, Lower Colorado River	Departures from cumulative sums of Column (14)	Cumulative depletion of Lake Mead
			Actual	Theoretical Future											
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1938	15.4	117.3	2.3	7.6	10.1	69.5	15.6	120.7	0.2	0.0	3.9	13.8	14.2	....	....
1939	9.4	107.5	1.8	6.2	5.0	61.4	9.6	110.9	0.2	−8.1	6.2	5.0	4.2	....	....
1940*	15.0	109.4	2.3	7.5	9.8	63.0	15.5	113.0	0.5	1.6	−0.6	17.9	18.4	....	....

* Values for 1940 are imaginary—believed based upon a discharge at Lee's Ferry, not likely to be reached.

value for 1940. Of most interest in this table is the indication that in 1940, to meet conditions previously explained, including the compact conditions in the presence of ultimate Upper Basin use of water, it would be necessary to permit to flow past Lee's Ferry all water draining from the Upper Basin during the year, plus an additional amount of 0.6 million acre-ft. This conclusion, based upon a rather large imaginary river flow assumed for 1940, seems fully valid because the prospect of a river flow in 1940 of as great as 15 million acre-ft at Lee's Ferry, the assumed value, seems quite unlikely at the present writing (May, 1940). River flow, coupled with men's attempts to read the future and provide for it, does seem very subtle.

The writer does not believe one can depart materially from the data of Tables 2 and 4 or the conclusions drawn therefrom while conforming to the terms of the Colorado River Compact and to the generally accepted interpretations.

Of course one must admit the possibility of other dispositions of Colorado River water. Large quantities of water may be absorbed in Mexico by appropriation sustained by international courts or by treaty grants. Arizona, not subscribing to the Compact, may legally appropriate a large quantity of water prior to its appropriation elsewhere and find ways of financing construction of necessary works. However, the vast expenditures required for water-supply development in Arizona must apparently be financed by the Federal Government and approved by Congress. Therefore it would appear that large-scale developments made contrary to the stipulations of the Colorado River Compact would involve flagrant violation of trust by the United States Government which is a party to the Compact. On the other hand, the Upper Basin may not desire to construct, or may be unable to finance, works in the Upper Basin and therefore water could be used elsewhere without protest. These and perhaps other possibilities exist, but they are not in accord with the preponderance of the opinions with which the writer is familiar.

The peculiarities of water distribution imposed by the Compact during drought periods appear to require an eventual modification of Compact provisions.

Referring to Table 1, which indicates the prospective water supply to be drawn reasonably soon from the main Colorado River below Lee's Ferry, the writer of necessity relied upon his own judgment to sort out acceptable items from the considerable quantity of discordant data. It again may be emphasized that an effort was made to be guided more by what is on the ground or posted on financial books and relatively less by statements of intentions.

Further comment may be made with respect to the Gila Project which was considered in Items 5 and 6 of Table 1. The construction of irrigation works for the first unit of this project, approximating 150,000 acres, is well advanced (May, 1940). So far as is known, there are not definite plans with respect to the time of construction and financing of additional works for other units of this project. The use of the term "Gila Project" by the Bureau of Reclamation is limited to an area of about 500,000 acres mentioned in the explanation accompanying Table 1. The value 2,900,000 acres was placed in Table 1 to indicate the extent of what some consider the possibilities of irrigation development in the Gila Valley. (Proposals, large and differing in detail, for the irrigation of areas approaching 2,900,000 acres in the Gila Valley of Arizona, are occasionally referred to as the Gila Project.) The writer did not go beyond an annual allotment of 405,000 acre-ft from the Colorado River to the Gila Project because this is ample water to supply the unit of the project for which there is assurance that works for its irrigation will be built; and also because each further appropriation of water for continuing use in the Lower Basin appears to add to a total which is already close to contravening the partition provisions of the Colorado River Compact. One would likely hold that, under the Compact, the irrigation of 2,900,000 acres of land in the Gila Valley of Arizona would require all water reserved for Lower Basin use.

A discussion of the fundamentals usually offered in justification of subsidies to irrigation and other water development in the United States was not the

object of the paper. In the end, Government subsidization of water development or any other development is based on expediency, established policy, availability of money or credit, the adequacy of favorable sentiment, and other items. The final decision is made in political circles.

Unified judgment of the justifiableness of subsidization would be almost impossible to obtain. Even agreement of what constitutes subsidization seems impossible. Some will regard repayment of a Government investment in an irrigation project by 40 annual payments of 1/40 the first cost and without interest as complete repayment. (This is the present repayment plan of the Bureau of Reclamation—it governs repayment of advances for the construction of the All-American Canal which was built to convey Colorado River water to the Imperial Valley.) Others will compare this to a commercial investment which requires repayment in the same length of time, with interest at, say, 5%, and find the annual commercial repayment is 2.3 times the other. The latter, therefore, may conclude that the 40-yr repayment without interest is equivalent to approximately a 57% subsidy. If a Government rate of interest of 3% is taken, the subsidy which may be concluded is 42 per cent. The first WPA plan of financing (a portion of Parker Dam was financed in this manner) involved what is usually referred to as a 30% subsidy or "grant."

Referring generally to Government water-supply financing in the United States—after a general financing policy has been determined and applied, it has been true too often that subsequently "adjustments," "moratoria," and "commutation of payment" have intervened so that cost repayments have been considerably less than were originally anticipated. Members of the engineering profession of recognized high standing occasionally have supported these modifications as being entirely justified and sufficiently common so that they are regarded as a likely if not an inevitable consequence of Government financing and therefore that Government financing is tantamount to subsidization. Nearly all of the new water-supply construction involving approximately a half-billion dollars and forming part of the development of the Colorado River below Lee's Ferry has been financed by the Government, and concessions (for instance, reduction of the interest rate of the Reconstruction Finance Corporation to the Metropolitan Water District of Southern California from 5 to $4\frac{1}{8}$%) have already been granted. In April, 1940, a determined effort was being made before Congress to obtain legislation permitting a reduction in the interest rate applying to repayments of the Government's investment in the Hoover Dam from 4 to 3 per cent. The bill embodying this change seems to have the benediction of all concerned. It also provides for deferring the beginning of repayment of that portion of the cost of the Hoover Dam which was allotted to flood control for about 50 years. Some will regard this as a substantial measure of subsidy and back their arguments with financial computations.

Throughout the American West, there is general agreement that Government subsidization of irrigation development to the extent of the current Bureau of Reclamation (Department of Interior) plan of financing is justified. Few advocate subsidization beyond this. It is the policy now in operation. Literature stating the pros and cons is voluminous. There is no need for the writer to re-state the case or particularly support it.

In Mr. Elder's discussion, mention is made of some technical points of hydrography. One is the apparent excessive loss from Lake Mead (not necessarily all real loss). Present data and their analyses seem to support fully the existence of excessive apparent loss and to indicate that a portion of the loss is genuine. It is probable that aeroplane surveys, which were referred to, and more data from the operation of the reservoir, will help to decide the nature of losses.

A second hydrographic point is the probable frequency of major drought periods. In this connection it is generally becoming apparent, even to optimists, that the region is having one drought that is truly severe. Data regarding recurrence of major droughts are indirect or only inferential. The First Annual Report of the Metropolitan Water District of Southern California states:[8]

> "By analyses of old flood records, mission crop reports, miscellaneous notes, diaries of early settlers, and other indirect means, H. B. Lynch, consulting hydraulic engineer of Los Angeles, has extended these [meteorological] records with reasonable dependability back to the year 1769. The results show that there has been no change in mean climatic conditions in Southern California since that date at least, although drought periods are indicated that were both more severe (1822–32) and of longer duration (1842–83) than any that have occurred during the present century."

A third hydrographic point is the amelioration possibility of additional large reservoirs. This is a possibility, but the beneficial effects of reservoirs are strained by the evaporation at the water surface and the long hold-over storage required in connection with the Colorado River. The writer would emphasize in this connection, and in some others regarding the hydrography of the Colorado River (or other rivers), that statements of general effects of hydrographic components on the solution of hydrographic problems may be misleading. In place of dependence on such statements it is recommended that an analysis be made by suitable periods, and for each, to apply the conditions with respect to withdrawals and losses of water as they would occur and credit accretions of water provided by natural conditions and the conditions provided by judicious management of available water resources. All must be conducted with due regard to the fact that at any assumed time no extensive knowledge of future conditions is available other than general knowledge indicated by past experience. Pronouncements of how river discharge is to be regulated or distributed (based upon knowledge of average flows and unsupported statements of how reservoirs affect stream regimen) should be subjected to careful scrutiny.

A fourth hydrographic point is the prospective quantity of water that will be lost at or below Imperial Dam. In the preparation of Table 1 a rather low value for this loss was selected from among several authoritative statements of much larger values. Included in the losses in question, in addition to that required for silt sluicing, are: Incidental flood spill, return drainage water allowed to flow to waste because of high salt content, inevitable wastage from irrigation canals, and possible wastage of part of the power water on the Yuma Project or on the All-American Canal near the heading of the Imperial Canal.

[8] First Annual Report of the Metropolitan Water District of Southern California, 1939, p. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

AN IMPROVED METHOD FOR ADJUSTING LEVEL AND TRAVERSE SURVEYS

Discussion

By Clarence Norris, Esq., and Julius L. Speert, Assoc. M. Am. Soc. C. E.

Clarence Norris,[17] Esq., and Julius L. Speert,[18] Assoc. M. Am. Soc. C. E. (by letter).[18a]—The comments of those who have discussed the paper have added vital information to it and have greatly increased its value to surveyors. A number of questions have been raised, also, which justify further discussion.

The writers cannot conceal their gratification at the unanimity with which most of those who have discussed the paper welcome the new method of adjustment. Although the method has withstood the test of use by the computers of the U. S. Geological Survey, as mentioned by Mr. Wilson, and its merit has been fully demonstrated, it was feared that its simplicity might be so hidden by the complex instructions required to present it that its real value might be lost.

There can be no question that, at first glance, the new method appears complex, and it was anticipated that the claims that it "is so simple as to be easily understood by any one familiar with the elements of surveying and of algebra" and that "a complete adjustment can be made by a clerk familiar with the use of the computing machine" would provoke disagreement. However, the experience of the Tennessee Valley Authority (TVA), as presented by Messrs. Whitmore, Miner, and Byrd, that the method was used satisfactorily "even by freshmen cooperative students who have had no previous computing experience," is a strong argument in support of these claims. The ease with which the method can be mastered is further attested by the experience of the Geological Survey. During the past few years, several computers who have come into the office have grasped the method apparently without difficulty.

Note.—This paper by Clarence Norris, Esq., and Julius L. Speert, Assoc. M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Messrs. George H. Dell, and Howard S. Rappleye; February, 1940, by Messrs. R. M. Wilson, Cleveland B. Coe, and George D. Whitmore and C. C. Miner and W. O. Byrd; and March, 1940, by Robert C. Sheldon, Assoc. M. Am. Soc. C. E.

[17] Asst. Topographic Engr., U. S. Geological Survey, Washington, D. C.

[18] Asst. Topographic Engr., U. S. Geological Survey, Washington, D. C.

[18a] Received by the Secretary April 22, 1940.

These men have included recent engineering graduates, a few college graduates with a knowledge of mathematics but no training in engineering or surveying, and a few older field men who have had relatively little mathematical background or have been out of school so long as to have lost all familiarity with it. In each instance, the mechanics of the adjustment were so readily acquired as to justify the writers' claim of simplicity.

In spite of all this, it was not the writers' intention to recommend that adjustments be left entirely in the hands of a clerk. However, it is believed that, under competent engineering supervision, the mechanics of the adjustment can be performed quite satisfactorily by untrained personnel.

The slide-rule solution of the equations suggested by Mr. Dell appears to be an excellent substitute for the Doolittle solution in those adjustments for which slide-rule precision is adequate, particularly for the adjustment of a level net. For a traverse adjustment, however, the entire process would have to be performed twice, and it appears that a single solution by the Doolittle method, even by slide rule, might be quicker.

Mr. Dell's recommendation that the Doolittle method of solving equations be incorporated in the mathematics requirements of engineering students suggests the desirability of including the theory of this method of adjustment in the surveying courses of such students. Because of its complexity and mathematical background, the method of least squares is not, so far as the writers are aware, taught to engineers except in advanced courses in geodesy or mathematics. The principle of weighted means, however, is so basic and simple that even a student of elementary surveying should be able to grasp it and appreciate its significance. Having once absorbed the theory of adjustment, even if the mechanics of the solution have been long forgotten, an engineer should have relatively little difficulty in making a simple adjustment if called on to do so in later years. Then the predicament described by Mr. Coe of searching "many texts without success for a simple method of adjustment for a series of interlocking traverses" could, to an extent, be avoided.

Messrs. Whitmore, Miner, and Byrd, like several of the others, recognize that results by the new method are identical with those by the conventional least-squares methods. Nevertheless, they seem to limit its use by the TVA to surveys of third-order limits or lower, and for first-order and second-order levels continue to use the longer methods described by the U. S. Coast and Geodetic Survey. The writers wonder why this should be so. Although the new method is not recommended for first-order traverse, it is fully adequate for adjusting the most precise levels. It must be emphasized that the new method is equally as "exact" as the conventional least-squares methods, the only difference being that the approach to the normal equations has been "streamlined."

They also suggest that an inverse computation be made to correct the azimuth and length of each course to agree with the adjusted coordinates of its ends. For this purpose, the writers propose the following modification of the formulas used in correcting triangulation lines after readjustment of datum:[11]

$$da = K\,(dE \cos a - dN \sin a) \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

[11] "Readjustment of Triangulation Datum," by Julius L. Speert, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 1002.

and

$$ds = dN \cos a + dE \sin a \ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

In these formulas, dN and dE are the adjustment corrections at the beginning of a course, A, minus those at the other end, B, in feet; K, in seconds, is $\frac{1}{s \sin 1''} = \frac{206{,}300}{s}$; K, in minutes, is $\frac{1}{s \sin 1'} = \frac{3{,}438}{s}$; a is the observed azimuth from A to B; s is the measured length of the course, in feet; and da and ds are the corrections to be applied to a and s, respectively, to bring them into agreement with the adjusted coordinates of A and B. The quantities da and ds will always be so small that slide-rule computation will afford ample precision. If a number of lines are to be adjusted by these formulas, preparation of a simple computing form would help to standardize the work and save time.

Mr. Dell makes the interesting observation that, in this method, at least one station in the survey must be treated as a fixed point. In an isolated survey, where no "fixed point" is available, any one station may be assigned an arbitrary adjustment correction (preferably zero), and all other adjustment corrections may be referred to that assumed datum. If, later, a more satisfactory datum is established, the entire system may then be shifted by a uniform correction to fit the new datum.

A somewhat similar interesting problem recently came to the attention of the writers. A third-order traverse net was tied at several places to an old first-order traverse line and also to several triangulation stations. The triangulation was on the 1927 North American Datum, but the traverse line was on an old datum and, with the data available, could not be brought to the 1927 datum for the purpose of controlling the adjustment of the third-order net. It was desired to adjust the new traverse to the 1927 North American Datum, and still give the first-order line all the influence in the adjustment to which it was entitled. In other words, the first-order line could not be used to control the datum of the new traverse, but relative positions along its route were to be held as fixed by its original adjustment. Thus, the old line was free to be translated, but could not be distorted by the new adjustment. This was accomplished in the adjustment by treating all ties to the first-order line as if made to a single junction, this junction to receive an adjustment correction the same as any other in the third-order net. This had the effect of including the first-order line in the adjustment, and of giving its sections infinite weight, but at the same time allowing it perfect freedom in shifting its datum. By this procedure, all ties to the first-order line received the same datum correction and were brought to the 1927 North American Datum, and no distorting adjustment corrections were thrown into the parts of the first-order line.

Mr. Rappleye suggested that a sample of a level-net adjustment should have been included in the paper. It is hoped that the adjustment presented by Mr. Coe will fill that need. Although Mr. Coe's example may not be truly a "net," it has all the elements of a larger adjustment and fully illustrates the principles. In this connection, a few points in Mr. Coe's rules relating to Table 11 should be clarified. In the solution of the equations, the N- and S-columns must be carried down simultaneously with the body of the table in order to provide a check on each line before proceeding with the next line. Otherwise, a mistake

in the solution may pass undetected and affect some of the subsequently computed values. In other words, rules 10 and 11 should follow immediately after rule 2. Similarly, rules 8 and 9 should be combined with rule 6, emphasizing that the *c*-factor used is always taken from the column of the principal term of the equation being formed.

Mr. Dell raises a vital point concerning the importance of careful determination of weight factors for the lines of a survey net, and Table 7 gives striking evidence of this importance. As brought out by Mr. Dell and several others, the proper weight of a line is a very complicated function of a large number of uncertain, and at present unmeasurable, factors. It might even appear, at a casual glance, that careful adjustment by any "precise" method is an unwarranted refinement so long as such uncertainty exists in the determination of weights. The problem of weights, however, enters into all methods of adjustment in about the same way, so that, strictly speaking, the matter of determining weights is not involved in the method used in making the adjustment. It appears that, whether weights are assigned in accordance with the simple rules suggested or whether a more complex procedure is used to include other factors involved, a reasonable adjustment may be expected if all the elements of the survey are properly balanced against one another. The importance of including all the elements of a survey in a general adjustment, rather than adjusting line by line, is illustrated by the example presented by Mr. Sheldon. Nevertheless, the writers agree that the determination of weights is perhaps the weakest factor that enters into an adjustment, and a practical means of evaluating all the uncertainties involved is much to be desired.

The wide applicability of this method of adjustment is illustrated by the variety of problems mentioned in the discussion. The traverse example in the paper was presented in terms of feet of northing and easting; the traverse of the Geological Survey, described by Mr. Wilson, is computed and adjusted in terms of degrees, minutes, and seconds of latitude and longitude; the surveys of the Panama Canal system, discussed by Mr. Sheldon, use degrees, minutes, and feet of latitude and longitude. Similarly, in levels, Mr. Coe's example is adjusted in foot units, and the adjustments mentioned by Mr. Rappleye are in metric units. Inasmuch as the units of measurement enter the adjustment only in the absolute columns, and as the absolute columns of an adjustment are not interdependent, it is not even essential that the same units be used in both columns. Thus, the fact that a second of latitude is different in length from a second of longitude does not interfere with a simultaneous adjustment of both. Similarly, under proper circumstances, seconds of latitude and longitude and feet or meters of elevation could be included in the same adjustment.

In conclusion, the writers wish to express their sincere gratitude to all those who have discussed the paper for the interest they have shown and for the valuable suggestions and comments offered.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EFFECTIVE MOMENT OF INERTIA OF A RIVETED PLATE GIRDER

Discussion

By Messrs. Charles U. Kring, Thomas C. Shedd, and Scott B. Lilly and Samuel T. Carpenter

Charles U. Kring,[37] Jun. Am. Soc. C. E. (by letter).[37a]—Apparently, the authors presuppose that the theoretically correct method of designing a plate girder is by use of the moment of inertia, and that the crucial question is: "What moment of inertia should be used?" No mention is made of the widely used "flange-area method," probably because they regard it as only an approximate makeshift. For finding the actual stress in the extreme fibers, that is undoubtedly true, but, for that purpose, so is the moment-of-inertia method only a makeshift, since, as the authors point out, the actual stress in some points of the flanges will have reached the yield point when the maximum stress as found by the moment-of-inertia method is only 18,000 lb per sq in.

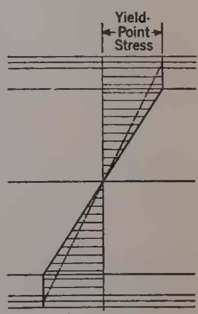


Fig. 11.

If it is agreed, then, that the ultimate aim of a method of design is not to produce a girder that will have a certain stress in the extreme fibers under the design load, but to produce a girder that will have a certain factor of safety under that load, then the writer believes it can be shown that, theoretically, the moment-of-inertia method is not as correct as the flange-area method.

A girder that is properly designed and properly braced against lateral buckling will fail by excessive deflection caused by plastic yielding of the flanges. The deflections

Note.—This paper by Scott B. Lilly, M. Am. Soc. C. E., and Samuel T. Carpenter, Assoc. M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1939, by Messrs. William R. Osgood, Clyde T. Morris, B. R. Leffler, E. Neil W. Lane, Lewis E. Moore, W. E. Black, and L. E. Grinter; January, 1940, by Messrs. Charles M. Spofford, E. Mirabelli, Edward Godfrey, Walter H. Weiskopf, and C. D. Williams; February, 1940, by Messrs. Alvin B. Auerbach, Jonathan Jones, and R. L. Moore; March, 1940, by Messrs. J. M. Garrelts and I. E. Madsen, Francis P. Witmer, and A. W. Fischer; and May, 1940, by Messrs. J. R. Shank, and Harold D. Hussey.

[37] Fellow, Civ. Eng., Univ. of Illinois, Urbana, Ill.

[37a] Received by the Secretary April 15, 1940.

[38] "Theory of Limit Design," by J. A. Van den Broek, M. Am. Soc. C. E., *Proceedings*, February, 1939, p. 193.

will not be excessive, however, until the load is such that plastic yielding has occurred throughout the entire cross-sectional area of the flange.[38] Under that load, the ultimate load, the stress distribution will be as shown in Fig. 11. Under any less load, the stress in some fibers of the cross section of the flange will still be within the elastic range, and the deflections of the girder will still be of the magnitude of elastic deformations. Any additional load would cause the web to wrinkle and the girder to fail due to excessive deflection.

It is evident, then, that if a girder is designed by the flange-area method in which the allowable stress is assumed to be distributed uniformly over the entire flange area (net area in tension flange) the true factor of safety will be the yield-point stress of the material divided by the allowable stress. If 30,000 lb per sq in. were the yield-point stress and 18,000 lb per sq in. the allowable stress, the factor of safety would be 1.67. "Factor of safety" is defined herein as the ratio of the ultimate load to the design load. What the actual stress is at any point in the flange is not known, any more than it is when using the moment-of-inertia method, and, except where fatigue may be a factor, it is not important.

The flange-area method is slightly in error in regard to the addition of one-sixth the web area (one-eighth for tension flange) to the flange area, since under ultimate load it assumes the distribution of stress in the web to be as shown by the dotted line in Fig. 11. This error, however, is not only on the safe side (as an examination of Fig. 11 will show) but is also so small as to be negligible, except possibly in very shallow, heavy girders. Where the moment-of-inertia method is used, the factor of safety, even with the same allowable stress, will vary, being larger for shallow girders than for deep girders (as shown by the stress-distribution diagrams), and in no case will it be the yield-point stress divided by the allowable stress as is commonly assumed.

From consideration of the foregoing, it is believed that the moment-of-inertia method is not only less theoretically sound than the flange-area method, but that the latter also satisfies the authors' requirement that "design methods should be as simple as possible."

Thomas C. Shedd,[39] M. Am. Soc. C. E. (by letter).[39a]—The first two sentences in the authors' introduction are: "It is customary for structural designers to consider a net moment of inertia in the design of plate girders. For the purpose of this paper, 'net moment of inertia' will be defined as the moment of inertia computed by deducting the effect of the material removed by holes in the tension flange." These sentences imply two things: First, that, in general, structural designers proportion plate girders on the basis of the moment of inertia; and, second, that in doing so they deal with the net moment of inertia as defined by the authors. There seems to be little basis in fact for either implication.

The writer always has been greatly interested in specifications for the design of structural steel bridges and buildings, and has studied minutely the changes and developments that have taken place since the first formal specification was issued by Clarke, Reeves and Company (later the Phoenix Bridge Company)

[39] Prof., Structural Eng., Univ. of Illinois, Urbana, Ill.

[39a] Received by the Secretary April 29, 1940.

on March 1, 1871. As a result the writer thinks it is the exception rather than the rule for structural designers to consider the moment of inertia of a girder, and even more exceptional for them to consider net moment of inertia as defined by the authors.

The fad for requiring the proportioning of plate girders by the "moment of inertia method" is a relatively recent and entirely unnecessary development. Quotations from some well-known specifications will give some clue as to when that development appeared.

The specifications issued by the American Railway Engineering Association in 1910 were widely used, and even more widely patterned after in the writing of other specifications. In that specification may be found:[40]

> "Plate girders shall be proportioned either by the moment of inertia of their net section; or by assuming that the flanges are concentrated at their centers of gravity; * * *."

These specifications represented general practice until they were superseded by the 1920 specifications, issued by the same organization. The 1920 specifications stated:[41]

> "Plate girders shall be proportioned either by the moment of inertia of their net section including compression side; or by assuming that the flanges are concentrated at their centers of gravity."

A specification prepared and presented in 1923, by a committee of the Society, included the clause:[42]

> "Plate girders shall be proportioned by assuming that the flanges are concentrated at their centers of gravity, * * *."

A specification prepared by a joint committee of the Society and the American Railway Engineering Association, dated March, 1929, contained the statement:[43]

> "Plate girders shall be proportioned by the moment of inertia of their net sections, including the compression sides; or by assuming that the flanges are concentrated at their centers of gravity, * * *."

The original specification issued by the American Institute of Steel Construction contained the forerunner of present-day clauses in the form:[44]

> "Plate girders with webs fully spliced for tension and compression shall be so proportioned that the unit stress on the net section does not exceed the stresses specified in section five as determined by the moment of inertia of the net section."

It will be noted that this clause does not require that girders be proportioned by the so-called moment-of-inertia method—merely that they shall be so pro-

[40] A.R.E.A. General Specifications for Steel Railway Bridges, Art. 29, 4th Ed., 1910.

[41] A.R.E.A. General Specifications for Steel Railway Bridges, Art. 116, 2nd Ed., May, 1923.

[42] Final Report on Specifications for Design and Construction of Steel Railway Bridge Superstructure, *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), Art. 323, p. 482.

[43] General Specifications for Steel Railway Bridges, *Proceedings*, Am. Soc. C. E., December, 1929, Art. 427, p. 2658.

[44] A.I.S.C. Standard Specification for Structural Steel for Buildings, June 1, 1923, Art. 7.

portioned that fiber stresses computed from the net moment of inertia will not exceed a specified value. That requirement can be met easily and accurately without computing either the net or the gross moment of inertia.

The first widely accepted specification to adopt a definite requirement that girders be proportioned on the basis of the moment of inertia, as far as the writer knows, was the American Railway Engineering Association specification of 1935 which stated:[45]

> "Plate girders and other members subject to bending that produces net tension on one face shall be proportioned by the moment-of-inertia method, using the net section of the compression side as well as of the tension side."

This clause was revised in 1938 to read:[46]

> "Plate girders, I-beams, and other members subject to bending that produces tension on one face, shall be proportioned by the moment-of-inertia method. The neutral axis shall be taken along the center of gravity of the gross section. The tensile stress shall be computed from the moment of inertia of the entire net section and the compressive stress from the moment of inertia of the entire gross section."

The writer can find little support, either in his experience or in an examination of specifications, for the authors' statements quoted at the beginning of this discussion. The definition of net moment of inertia given by the authors presumes a neutral axis resembling the edge of an old-fashioned bread knife. The most casual consideration of the flexed girder would seem to establish the plane of no stress as a continuous, smoothly curving surface which cannot depart more than an entirely negligible distance from the center of gravity of the gross section—it cannot conceivably "jitterbug" between the center of gravity of the gross section between holes and the center of gravity of the net section at holes.

To design plate girders by the moment-of-inertia method, as recommended by the authors, and required by many recent specifications, is an unnecessary waste of time and effort. It is an easy matter to proportion a girder by a simple modification of the commonly used "flange-area method," so that its extreme fiber stress, either on gross section or net section, will not exceed a specified value. One may proceed as follows:[47] Using the flange-area method, calculate the required net area of the tension flange in the ordinary way, multiply this area by the ratio of the estimated over-all depth to the estimated effective depth, proportion the girder, and check the estimated effective depth and the estimated over-all depth. If the estimated depths agree exactly with the actual values the girder as proportioned will have a calculated extreme fiber stress (using the moment-of-inertia method as defined in the specifications previously noted) not greater than the assumed value, and not less by more than 1% or 2% (usually less than 1%), except for very shallow and heavy girders, in which the discrepancy may be as much as 5%. If the differences

[45] *Bulletin No. 374*, A.R.E.A. Specifications for Steel Railway Bridges, February, 1935, Art. 426, p. 21 (Vol. 36, p. 653).

[46] *Manual*, A.R.E.A. Specifications for Steel Railway Bridges, Art. 426, p. 15–15.

[47] "Structural Design in Steel," by T. C. Shedd, John Wiley & Sons, Inc., 1934, p. 178 (first published in mimeograph form in 1928).

between estimated depths and actual depths are considerable the designer will revise his calculations as he would in ordinary design. Obviously one may follow the same procedure if he prefers to design on the basis of gross area.

It should be clear that the flange-area method, when modified in the manner described,[47] would give exactly the same result as the moment-of-inertia method if effective depth were defined as the distance between the center of gravity of the compressive flange stresses and the center of gravity of the tensile flange stresses—the web area being translated into an equivalent flange area in substantially the usual manner.

In their conclusions the authors state that: "* * * the data from these tests point to the acceptability of the gross moment of inertia for design purposes." The writer is not sure he understands what is meant by the statement. If it means that one may compare girders he wishes to design with those which have been designed and built, with satisfactory results, on the basis of computations using gross moment of inertia, probably most engineers will agree, although they may not see any advantage in convenience or accuracy in so doing. If the statement is intended to convey the idea that one may apply the unit stress generally used on net section to gross section and secure a girder with the same margin of safety, the writer cannot agree—he finds no evidence in the data reported to support such a thesis.

SCOTT B. LILLY,[48] M. AM. SOC. C. E., AND SAMUEL T. CARPENTER,[49] ASSOC. M. AM. SOC. C. E. (by letter).[49a]—The interest in this paper, as manifested by the many discussions, indicates that the structural design of plate girders is in a state of flux. The varied points of view that have been presented are of great value to all interested in this problem, and will be followed closely in planning future investigations.

As was to be expected, criticism of the writers' conclusions has been voiced by many discussers. On the other hand, all of the discussers who had experimental data to present have agreed that the gross moment of inertia is acceptable for design purposes. Although this agreement was indicated, several of the discussers based their conclusion on reasons which differ from those stated by the writers.

In general the discussions presented reflect the traditional thinking of the structural engineer in regard to net section as applied to tension members. The writers do not advocate the discarding of net-section rules for tension members, as so many discussers imply, but they wish to emphasize the fact that a plate girder is a structure in which tension and compression members are combined. They limited the original paper to the action of the girder under static loads within the working range, and were interested in over-all stress conditions rather than in the measurement of high localized stresses at, and around, the holes. The tests have shown that these strains are not greatly affected by these highly stressed areas, and that for all practical purposes the strains might be computed by the gross moment of inertia. Therefore, it is believed that the gross moment of inertia will represent working conditions of

[48] Prof., Civ. Eng., and Chairman, Div. of Eng., Swarthmore Coll., Swarthmore, Pa.

[49] Asst. Prof., Civ. Eng., Swarthmore Coll., Swarthmore, Pa.

[49a] Received by the Secretary May 7, 1940.

the girder more accurately than the net moment of inertia, since no matter what method of computation is used the real value of the high stress around the hole will be little affected. As several discussers have mentioned, these stress concentrations will be of importance if fatigue is a problem.

The tests also indicated that the over-all strains in the two flanges of the girder were practically the same, showing clearly that if the net-section advocates wish to be entirely right and consistent, they should deduct the holes from the compression flange as well as the tension flange. This is an elaborate statement of the fact that the stress concentrations exist in both flanges and that the rivets in the compression flange are no more effective in the so-called filling of the hole than those of the tension flange.

Although the writers restricted their paper to the action of a plate girder under static loads within the working range, the discussions indicate that the problem cannot be so limited. The discussions focus the attention on the compression flange rather than on the traditional tension flange. The writers often discussed failure by buckling while making the tests, but did not touch upon the subject in their paper. They regret this omission, but it has been ably discussed by several of the contributors, notably Mr. Hussey. Although the writers believe that their tests at working loads were sufficient to justify the use of the gross moment of inertia, a consideration of the stability of the compression flange makes more acceptable the use of the gross moment of inertia in design. This is true because in a failure by buckling a considerable portion of the girder is involved rather than a single cross section, and the gross moment of inertia tells more about the behavior of the entire structure.

The majority of the discussers seem to indicate that the use of the gross moment of inertia for determining deflections is common practice and such use is not questioned. The writers believe that this phase of the problem requires no more discussion, except to point out that in unusual cases Equation (5) is available to give a more accurate value of the effective moment of inertia.

The tests reported show that for all practical purposes the neutral axis may be considered at the centroid of the gross section, and that there is no indication of an undulating elastic line. It is true that a discontinuity exists in the stresses around the flange holes, and it is also apparent that conditions are the same in both flanges, giving a symmetrical cross section in this respect. By the principle of St. Venant, the boundary discontinuities will have no bearing on stresses normally expected in the interior of the web plate, and hence cannot affect the position of the neutral axis.

Neither the net moment of inertia, the gross moment of inertia, nor any other of the proposed methods of design gives the actual stresses in and around the holes of a plate girder. When the stress concentrations around such holes are given careful study, it becomes evident that no simple method can be used to evaluate them. They usually reach the yield point stress at the edge of the hole, and they cannot be modified by the methods of computation. The thought that a unit stress computed from a net moment of inertia gives a clearer idea of the safety of the plate girder is fallacious because plate girders do not fail by the rupture, or the tearing apart, of the tension flange.

If a high load is placed upon the girder, all parts of the tension flange will reach the yield point stress. Certainly then, if ever, the net section should govern because all stress concentrations have been wiped out. The compression and tension flanges will reach the yield point stress conditions at the same time. The tests made by the American Bridge Company, and reported by Mr. Hussey, show what happens when this condition is reached. The compression flange buckles and the plate girder fails—a functional failure—and the stresses and the stress concentrations in the tension flange are not vital factors. That is, these stresses did not measure the safety of the girder, so the traditional emphasis of the designer upon stresses computed in the tension flange by net-section methods has had no real bearing upon the safety of the girder. All of the tests reported by the discussers confirm these conclusions of the writers that safety does not lie in the computed stresses in the tension flange.

Mr. Osgood calls attention to an article by Friedrich Hartmann,[5] which should be read by every one interested in this subject.[5a] Professor Hartmann states:

> "The taking into account of the weakening effect due to rivets in those parts of a girder that are under tension was a matter of course so long as the static failure of these portions was considered as being the controlling factor in regard to safety. However, a girder becomes unfit for service much sooner, in general, when non-permissible deformations occur."

After a theoretical discussion calculated to give the basis for the foregoing statement, Mr. Hartmann describes the test of a pair of newly fabricated riveted girders made of open-hearth steel. These girders had a span of 10 m (32.81 ft), a depth-to-span ratio of approximately 0.1, and flanges of angles and cover plates. They were tested to destruction and failed by the buckling of the compression flange. The stress in the compression flange at failure slightly exceeded the compressive yield point of the material. Verification of this latter statement is noted in the tests reported by Mr. Hussey.

As stated, Mr. Hussey presented the data of a series of tests which bear directly on this problem. The plate girders were loaded to failure, and in every case except one the tension flange showed no distress and the compression flange failed by buckling. The one failure in the tension flange which Mr. Hussey reported occurred in a girder where the compression flange had twice the area of the tension flange. In general these tests supplied the data at failure requested by several of the discussers and bear out the statement of Mr. Hartmann that the safety of the girder is not controlled by the tension flange.

Professors Morris and Shank suggest methods of averaging the net sections with the gross sections to obtain flange stresses and a weighted moment of inertia, respectively. The writers believe that an average was obtained experimentally by using a 10-in. strain gage, since such measurements over the rivets (or holes) actually integrate the existing conditions.

Mr. Lane has expressed, very clearly, a thought that was uppermost in the minds of the writers, while they were planning the tests, when he stated:

[5] "Sollen genietete Träger mit Berücksichtigung der Nietschwächung berechnet werden?" von Friedrich Hartmann, *Der Stahlbau*, Vol. 12, February 17, 1939, p. 25.

[5a] A translation into English has been placed in Engineering Societies Library, 29 West 39th Street, New York, N. Y.

"* * * riveted plate girder flanges will always consist of many small areas (adjacent to rivets) of high stress restrained and stiffened by surrounding areas of low stress." Highly stressed areas of this type have never given trouble.

Mr. Black computed the stresses in the riveted section of a rigid frame whose make-up was the same as that of a plate girder, and found that the measured stresses agreed with those obtained by using the gross moment of inertia. Mr. Moore reported tests made on an aluminum plate girder, in which he discovered that the stresses between rivets were higher than the stresses in the net section. He found that stresses computed from the gross moment of inertia correlated very well with those obtained by the use of a tensometer having a 0.5-in. gage length.

Mr. Jones has stated clearly the reasons for the adoption of the gross moment of inertia by the Committee on Specifications of the American Institute of Steel Construction. However, the writers want it clearly understood that the tests were not planned to confirm any action of any committee. They were designed to give essential data, and if the results seem to favor the gross moment of inertia it is simply because the confirmation is there.

The report of Professor Shedd on his careful research into the specifications of plate girders for the past fifty years is particularly valuable. Whether or not readers agree with Professor Shedd that girders should be proportioned by the "flange-area method," all will admit that as long as stresses govern the composition of flange areas, and the common theory of flexure is accepted, the moment of inertia, either gross or net, must be the final resort of the designer. The writers agree with Professor Shedd that the neutral axis does not resemble the old-fashioned bread knife.

The interest in the problems of the plate girder, which has been demonstrated by the large number of discussions, will insure many more investigations. The writers had only the results of two "pilot" test girders to guide them in writing their paper, but they are of the opinion that those tests, and the stimulating discussions, will go far in guiding future investigations, and hence will serve their major purpose. The writers are quite convinced that future investigations will be focused upon the compression flange rather than upon the tension flange. These future tests must include all types of cross sections of girders, varying degrees of restraint of the compression flange, and different depth-to-span ratios.

The writers close this discussion with the hope that each engineer will avail himself of any opportunity to test plate girders under working conditions and that he will also see to it that the results of his tests are given to the profession.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

THE RÔLE OF THE ENGINEER IN AIR SANITATION
A SYMPOSIUM

Discussion

BY MESSRS. GILBERT H. DUNSTAN, AND CHARLES LUNDY POOL

GILBERT H. DUNSTAN,[24] ASSOC. M. AM. SOC. C. E. (by letter).[24a]—So much levelopment in the field of sanitation and public health has occurred during the past decade that this Symposium is of great value in focusing attention upon one important phase. Professor Phelps has mentioned the pollution of the outdoors atmosphere, and its effect upon health. From the economic standpoint, such pollution may have other implications, one of which is flood control. Denuding hillsides of all vegetation by the fumes from smelters may require costly flood-control structures to prevent damage, in some cases even to the smelter itself. This is the case in Kessler Canyon, Utah.[25]

Mr. Bloomfield has made a valuable contribution by indicating the method of attack in an important problem of industrial hygiene. Engineers must take an increasing interest in this type of work. Mr. Bloomfield quotes the duties and qualifications of an industrial hygiene engineer, which were adopted by the Committee on Industrial Hygiene of the State and Provincial Health Authorities. These indicate graduation in chemical engineering as a qualification, followed by at least two years of graduate work in industrial hygiene, plus three years of experience. The writer would raise the question of whether or not a somewhat better sequence of studies might not be arranged if the man graduated in sanitary engineering and followed this by two years of postgraduate study. In any case, the breadth of training required seems to indicate at least six years of formal training.

Certainly the two terms "sanitary engineering" and "public health engineering" may be considered to be more or less synonymous; yet there seems to

NOTE.—This Symposium was published in November, 1939, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: February, 1940, by Messrs. Theodore Hatch, Donald Francis Griffin, Gordon M. Fair, and H. G. Dyktor.

[24] Asst. Prof. of San. Eng., Univ. of Alabama, University, Ala.

[24a] Received by the Secretary March 15, 1940.

[25] "The Barrier System of Flood Control," by L. M. Winsor, M. Am. Soc. C. E., *Civil Engineering*, October, 1938, p. 675.

be a tendency to give a slightly different meaning to each of them. Recent studies of literature relating to training in these fields seem to show a tendency to consider the sanitary engineer as the one who is concerned chiefly with water supply and treatment, sewerage and sewage treatment and disposal systems, and stream pollution. The public health engineer is concerned with these factors largely in a supervisory way; but in addition he is interested in rural sanitation, supervision of food supplies, and many other problems of a somewhat minor nature, involving engineering to some extent. He must deal with the public to a great extent, and his work is more closely related to that of the medical profession. The industrial hygienist is closer to the public health engineer than to the sanitary engineer under this distinction. Only one institution grants degrees designated in these two fields, and it is the writer's belief that the foregoing distinction is indicated by the two curricula as offered. However, at the present time the two terms are used loosely, and only time will tell whether any real distinction between the two fields will develop.

Charles Lundy Pool,[26] M. Am. Soc. C. E. (by letter).[26a]—In common with others competent to judge, the authors of this Symposium agree on certain fundamentals. A vast amount of new, useful knowledge is available to the sanitary engineering profession, the application of which promises both economic and public health benefits.

Some engineers sound a warning that the individual must beware lest attempted expertness in the fields of sanitation of waters and hygiene of air lead to accomplishment in neither. They recognize, however, that the profession is not limited by the limitations of its individual members and can effectively embrace diversified fields of endeavor without this danger, provided the fields are related. Others express the principle that in many cases the engineering approach is the only one to set a standard for improved conditions. Advances which new techniques make possible are often put into effect to better man's environment, although their relationship to his well-being cannot be measured in the maze of variables present, not the least of which is variable susceptibility of the individual.

Mr. Bloomfield testifies to the fact (see heading "Trends in Industrial Sanitation") that within the past few years "current knowledge of the methods for the control of many health hazards in industry has reached the stage where it may be applied successfully." He also presents conclusive data to impress the need of application of the new knowledge. Professor Phelps' review of the branches of science which now have much to contribute includes air bacteriology which has enjoyed a renaissance under the "midwifery" of W. F. Wells. Sanitarians may now compare airs as to their relative contamination so as to supplement the present physiologico-thermal guides by bacteriological controls. Professor Phelps also states (heading, "Industrial Conditions"), "It is believed that the public health engineer must be prepared, with the necessary collaboration from the associated sciences, to assume ultimate responsibility and control in all such cases."

---

[26] Chf., Div. of San. Eng., Rhode Island Dept. of Health, Providence, R. I.

[26a] Received by the Secretary April 1, 1940.

Now, what can be done to harness all this benevolent sporadicity of which the foregoing examples are but indicators of a vast ground swell? Industrial air sanitation has made rapid strides the past several years, and has been integrated into the work of health departments. The latter is not true of air sanitation of communities or public buildings. The majority of state health departments now have industrial hygiene engineering services.

If stream-pollution abatement programs had been left to progress only by the "fits and starts" which common law or damage suits might have stimulated, without the coordination of programs by state sanitary engineers, the art of wastes disposal would indeed be in a sorry state today, and its practice worse. Air sanitation of public places likewise needs to be ordered into the program of all state departments of health. This is particularly true today in view of the evolution of these departments to encompass more and more all applications of sanitation whether subject to immediately demonstrable health significance or not.

The work of the sanitary engineering, and of the industrial hygiene engineering, services should be closely coordinated to effect this program. The limitations of the individual must not be allowed to impede the progress of the profession. There is no reason why these two synonymous groups of sanitary or hygiene engineering cannot learn to work together in the same manner as water works engineers and sewerage works engineers, or bridge engineers and highway engineers.

In each state department the engineering service should prepare itself to evaluate all the programs of all branches of environmental hygiene and sanitation in the state, no matter by what state or local agency these are conducted. In this manner annual engineering reports to Commissioners of Health could be made comprehensive for the place which sanitation plays and should be made to play in the state economy. The administrative review of the status and adequacy of current endeavors need not require an expert qualified to apply the details of each branch of sanitation but, of course, must be conducted by one broadly grounded in the science of public health. Expert assistance in branch specialties may be had from the U. S. Public Health Service in the conduct of such evaluational surveys.

This general program is broached in this discussion on air sanitation because in many instances the topic could well be made the point of departure from an old lethargy. Of timely interest in the program into which air sanitation can fit is the following communication, addressed to all State Health Officers by the Surgeon General of the U. S. Public Health Service:

> "On February 1, 1940, a reorganization of the work of the Sanitation Section was effected with the object of transferring to that Section and its district engineer offices all environmental sanitation functions not already assigned to it.
>
> "The following units have been established, or will shortly be established, in the Sanitation Section: (*a*) Water Pollution Control (treatment of sewage and industrial wastes); (*b*) Rural and Suburban Excreta Sanitation (structures not connected to sewer systems); (*c*) Water Supply Sanitation; (*d*) Milk and Milk Products Sanitation; (*e*) Food Sanitation;

> (*f*) Garbage Disposal and Rat Control; (*g*) Recreational Sanitation; and (*h*) General Sanitation.
>
> "In accordance with the above reorganization the Milk Specialists and the Community Sanitation Consultants have recently been made responsible to the District Engineers, who are in turn responsible to the Chief of the Sanitation Section."

Note that the pattern is being set by the district engineers' offices of the federal service into the program of which state engineering services will need to be integrated. Note that "all environmental sanitation functions" are assigned to the Sanitation Section. The next unit that should be established beyond those listed, at first in industrial districts, should be one specifically for air hygiene. When the demand is felt there is no doubt but what this may be had.

Since state sanitary engineering functions are becoming so diversified it is of additional importance that federal district experts be stationed where they may be available on call to assist the state engineers in their salient duty of providing a clearing house for the advanced knowledge of all branches of sanitation. Especially is this true of expert advice on air sanitation in view of its newness and consequent imperfect dissemination.

It is most important that the entire sanitary engineering profession digest the significance of the trends and act as a body to influence and promote an ordered and purposeful evolution.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PROBLEMS AND TRENDS IN ACTIVATED SLUDGE PRACTICE

Discussion

BY E. SHERMAN CHASE, M. AM. SOC. C. E.

E. SHERMAN CHASE,[13] M. AM. SOC. C. E. (by letter).[13a]—A compilation of data relating to the design of thirty sewage treatment plants must have been a difficult and laborious task, and the author deserves the gratitude of sanitary engineers for his presentation of these data.

The non-uniform character of sewage, and the variable conditions under which its treatment occurs, make it necessary to use caution in the application of the results of laboratory research to practical problems of design and operation. For half a century the art of sewage treatment has been subjected to scientific investigation, and in spite of the mass of scientific research which has been conducted, sewage treatment continues to remain an art in which experience and judgment have at least as much weight as the researches of the laboratory. Therefore, design data such as presented by the author are of genuine value as an aid to the judgment of other engineers. However, such data need to be supplemented by the results of operation as given for nine of the thirty plants discussed by the author.

The increase in activated sludge plant capacity in the United States appears to be due mainly to the construction of a few large plants such as those at Chicago, New York, Cleveland, and Columbus. On the other hand, it is true that numerous smaller activated sludge plants have been built in recent years. Generally this type of plant is less practical for the small municipalities than for the larger cities, due to the complexity of the process and the need of relatively expert supervision. The smallest installation given in the author's tabulation is that at Ann Arbor, Mich., with a capacity of 4.5 mgd. Under certain local conditions activated sludge plants of even smaller capacity are sometimes more suitable than any other type, although it is usually advisable to install the simplest type of treatment for the small municipalities.

NOTE.—This paper by Robert T. Regester, M. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Langdon Pearse, M. Am. Soc. C. E.; and April, 1940, by Messrs. Frank C. Roe, and Gerard A. Rohlich and Clair N. Sawyer.

[13] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[13a] Received by the Secretary April 15, 1940.

At Leominster, Mass., where local conditions made it inadvisable to install an Imhoff tank-trickling filter plant, a small activated sludge plant has been in operation since 1938. The basic design data for this plant, comparable with the design data given in Table 1, are as follows:

Column No.	Description	Value
1	Location, Leominster, Mass.	....
	Nominal Capacity:	
2	Sewage flow, in million gallons daily	2.0
3	Human population, in thousands	25.0
	Year of Starting:	
4	Construction (WPA)	1934
5	Operation—preliminary treatment	1937
	activated sludge treatment	1938
6	Type of sewerage system	separate
	Preliminary Treatment:	
7	Coarse screen openings (cleaned mechanically), in inches	1
8	Grit chamber	yes
9	Pre-aeration	no
10	Number of mechanically cleaned settling tanks	2
11	Sedimentation period, in hours	1.5
	Aeration:	
12	Mixed liquor detention period, in hours	5.5
13	Returned sludge (percentage)	25
	Final Sedimentation, Mixed Liquor:	
14	Settling rate, in gallons per square foot daily	800
15	Detention period, in hours	2.2
16	Further treatment	none

From this tabulation it will be seen that the Leominster plant is more or less comparable, as regards design data, with the plants listed in the paper. The sludge is digested in a separate digestion tank and dewatered on open sludge beds. Since its initial operation the Leominster plant has been operated successfully with no more than the usual difficulties encountered in starting a new plant. The removal of biochemical oxygen demand and suspended solids has averaged about 90%. At times bulking has occurred in the final settling tank, which appears to take place when the strength of the sewage is greatest and to disappear when the sewage is weak. Bulking continues to remain something of a mystery and at Leominster the sludge index is not a wholly reliable criterion as to whether or not bulking is likely to occur.

There is some question in the writer's mind as to whether the biochemical oxygen demand test alone should be relied upon to show the degree of purification effected. For example, at Leominster the 1940 tests made by the State Department of Public Health showed an oxygen demand removal of 89.2%

as compared with removals of 72.3% and 71.2% for albuminoid nitrogen consumed. The fact that there is more than one stage for oxygen demand indicates that perhaps the 5-day oxygen demand removals do not tell the entire story and that some of the older tests are still of value as indexes of purification.

Certain innovations suggested by the author, such as tapered aeration and the use of magnetite filters for polishing activated sludge effluents, seem to be, as yet, of unproved value. As regards aeration, it is evident that it serves two purposes: First, agitation of the mixed liquor; and second, the supplying of oxygen. The amount of oxygen present in the air applied, in quantities sufficient to provide agitation, is more than enough to maintain aerobic conditions in aeration tanks under ordinary conditions. The problem of aeration involves not so much the matter of adequacy of supply as it does of getting the air into the mixture sufficiently rapidly. From the theoretical standpoint it would appear somewhat more logical to taper the loading rather than to taper the air, particularly as the time element is an important factor as regards oxygen absorption.

In conclusion, the writer wishes to emphasize his personal conviction that the designer should consult the operating staffs of treatment plants at every opportunity in order that his designs may embody the benefit of practical operating experience.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

TRANSIENT FLOOD PEAKS

Discussion

BY WALTER J. WOOD AND MAXWELL F. BURKE,
ASSOC. MEMBERS, AM. SOC. C. E.

WALTER J. WOOD,[49] AND MAXWELL F. BURKE,[50] ASSOC. MEMBERS, AM. SOC. C. E. (by letter).[50a]—An interesting treatment of a subject not new, but still controversial in many respects, is presented in the paper. Debris flows are of particular interest to engineers in Los Angeles County because of potential hazards to life, property damage, and silting of flood control and conservation reservoirs and debris basins.

The use, as a unit for comparison, of the "cross-sectional area per square mile of catchment area" is well chosen. Comparison by the usual method of discharge in cubic feet per second, or in second feet per square mile, is inadvisable in the case of this flood due to the almost complete lack of factual data on velocities and other hydraulic properties taken during the peak of the flood. The absence of gaging stations, except in Haines Canyon, and the inability of hydrographers to travel over flooded roads precluded obtaining flow data. Numerous lay estimates were furnished by various observers but they varied widely. The many flow-section areas obtained by the author are probably reliable for comparison of surges but certainly would not be reliable for use in the computation of discharge.

Although the rainfall conditions described by the author may produce the cross-sectional areas of flow measured by him, wave formation may occur even if it is unaccompanied by a sudden rainfall increase. The formation of waves, termed "traveling waves" or "slug flow," can increase the cross-sectional flow area to many times that occupied by the average flow. This is demonstrated by the observations reported by W. H. Holmes,[51] Assoc. M. Am. Soc. C. E., in 1936. The flow observed on January 5, 1935, at Mountain Street in the

NOTE.—This paper by Henry B. Lynch, M. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Ivan E. Houk, M. Am. Soc. C. E.; March, 1940, by Messrs. Gordon R. Williams, and Donald M. Baker; April, 1940, by Messrs. James M. Fox, F. C. Finkle, A. L. Sonderegger, and Harold C. Troxell and R. Stanley Lord; and May, 1940, by Messrs. Karl J. Bermel, and R. W. Davenport.

[49] Asst. Chf. Hydr. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

[50] Research Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

[50a] Received by the Secretary April 13, 1940.

[51] "Traveling Waves in Steep Channels," by W. H. Holmes, *Civil Engineering*, July, 1936, p. 467.

Verdugo Channel, was recorded by a water stage recorder about a mile and one half upstream. The average flow at the gaging station was about 1,100 cu ft per sec. This checks closely the flow computed from Mr. Holmes' estimates of hydrograph shape and his measurements of wave frequency; yet the maximum cross section occupied by the wave peak was $43 \times 8 = 344$ sq ft, compared with a flow cross section at the gaging station of 45 sq ft, a ratio of $7\frac{2}{3} : 1$. The cross section, slope, and construction of the channels at the two locations were practically identical. Assuming the velocity of water in the wave front to be equal to the observed velocity of the wave, the peak flow amounted to $43 \times 8 \times 23.5 = 8{,}100$ cu ft per sec; yet the peak flow at the gaging station above was not more than 1,500 cu ft per sec at the maximum. This represents an increase of 440% in peak flow due to channel hydraulics alone, without any benefit from rainfall variation, burned watershed, small catchment area, fan shape of tributary collectors, steep slopes, or any of the other factors assumed by the author to be contributing to flood wave production.

Similar observations made in concrete channels in the Montrose area during the flood of March 2, 1938, indicate that, for an average flow called 100%, the variation of flow rates was from 10% to 425% of this average due to the formation of these "traveling waves." These observations were made in the channels below debris basins, where variations in inflow rate had been smoothed out to very minor fluctuation in outflow rate. The water in these waves was not carrying debris other than silt in suspension.

During the storms of December 14, 1934, and January 5, 1935, observers were stationed in Pickens Canyon and Hall-Beckley Canyon to make observations on flows. Traveling waves developed in both these storms with waves occurring at intervals of 11 to 12 sec (between 7:58 a.m. and 8:36 a.m., in Fig. 21). Observations were made on random waves and a gage-height record

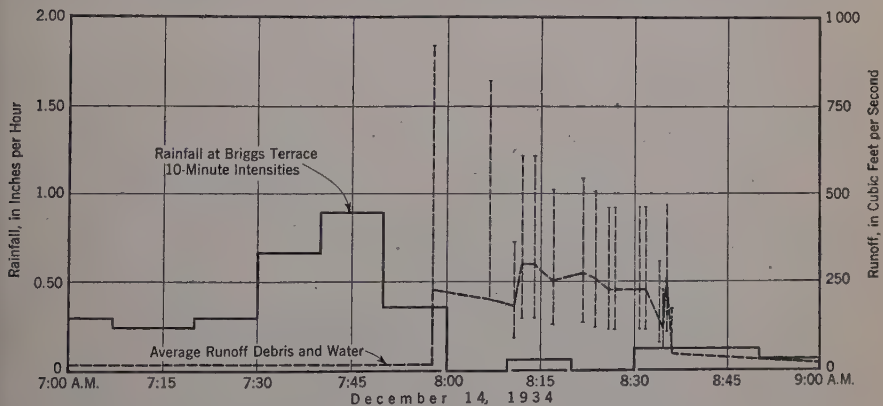


FIG. 21.—STORM OF DECEMBER 14, 1934, HALL-BECKLEY CANYON

was kept so that the average flow and the peak flow rates could be approximated. The hydrograph obtained from observations in Hall-Beckley Canyon during the storm of December 14, 1934, is shown in Fig. 21 as a sample of this

type of information. The vertical broken lines in Fig. 21 indicate random waves that were timed through the gaging reach. During this storm, in Hall-Beckley Canyon (drainage area, 0.77 sq mile burned) the maximum observed ratio of surge flow to mean flow was 925 cu ft per sec to 230 cu ft per sec, or 4 : 1. The maximum observed ratio of peak flow to minimum flow was 610 cu ft per sec to 150 cu ft per sec, again 4 : 1. At no time during the storm was the rainfall rate greater than 0.90 in. per hr for a 10-min period. During the January storm the maximum observed ratio of surge flow to mean flow was 2,110 cu ft per sec to 420 cu ft per sec, or 5 : 1. The maximum observed ratio of peak flow to minimum flow was 2,110 cu ft per sec to 140 cu ft per sec, or 15 : 1. At no time during the storm was the rainfall intensity greater than 0.95 in. per hr for a 10-min period.

For the January storm in Pickens Canyon, the maximum observed ratio of surge flow to mean flow was 1,050 cu ft per sec to 680 cu ft per sec, or 1.5 : 1. The maximum observed ratio of peak flow to mean flow was about 1,090 cu ft per sec to 600 cu ft per sec, or 1.8 : 1. The maximum 10-min rainfall intensity during the storm was 0.95 in. per hr.

This type of surge flow, or "traveling wave" flow, has been observed in Los Angeles County under practically every condition—with and without included debris, in natural channels and in artificial channels where normally Kutter's "n" value would vary from 0.014 to 0.035 or more, from mountainous drainage areas and from purely urban areas, for rectangular channels and for trapezoidal channels, for relatively large flows (4,500 cu ft per sec or more) and for very small flows—in fact, the only factor consistently present in slug flow phenomena appears to be that the velocity of flow must be supercritical. On the other hand, many flows at supercritical velocities do not produce this type of flow. Much remains to be learned about this phenomenon, and it seems to the writer to be too early to hazard an opinion as to the factors necessary and sufficient to produce this type of flow.

Debris production in the New Year's flood came largely from three sources:

(1) From the ground surface due to sheet and confined runoff over very steep slopes recently burned;

(2) From numerous land slides of large proportions, due to partial saturation of soil masses overlying bedrock at greater than the angle of repose (many of these, being adjacent to the channels, sloughed in and were "eaten" away by overflow after forming temporary dams); and

(3) From the channel bottoms where overlying soil, loose rock, and trees were carried away leaving clean bedrock exposed.

Some erosion of the debris cones in the valley area also occurred where the flows were partly confined. Total debris production was estimated at 659,000 cu yd.[4] If the source of debris is assumed to be only the burned area of 7.5 sq miles, a rate of production of nearly 88,000 cu yd per sq mile is obtained as the average. Rates from individual drainage areas were estimated to be as high as 100,000 cu yd per sq mile.

[4] "Los Angeles County Flood Control District Report New Year's Foothill Debris Flood, 1934," by E. C. Eaton, M. Am. Soc. C. E., report dated March, 1934, Chart No. 4 (not published).

Subsequent to the flood of January 1, 1934, seven debris basins were constructed in the area in addition to that at Haines Canyon and the Verdugo Debris Basin which was completed after the flood. The value of these basins was evident after the flood of March 2, 1938. These basins furnished the opportunity to obtain very good quantitative data on debris production from the same canyons discussed by the author. A close comparison cannot be made between debris production in the two floods, because the estimates for the 1934 flood were necessarily somewhat inaccurate due to the large proportion of debris deposited on streets and property, and also because an unknown quantity came from erosion on the valley floor below the canyon mouths. However, even a rough comparison is interesting.

Table 9 shows the results obtained from the March 2, 1938, flood, which produced 541,500 cu yd of debris. This came from the same drainage area

TABLE 9.—ANALYSIS OF DEBRIS DEPOSITED IN "MONTROSE AREA" DEBRIS BASINS FROM MARCH 2, 1938, FLOOD*

Debris basin	Drainage area, in square miles	Year watershed burned	Percentage of area burned since 1927	Debris deposited in basin, in cubic yards	Debris inflow to basin, in cubic yards per square mile	Cost of debris inflow, in dollars‡
Haines	1.53	1933	31	51,500	34,000	15,450
Dunsmuir	0.84	1933	100	58,500	70,000	17,550
Shields	0.27	1933	85	32,500	120,000	9,750
Eagle-Goss	0.61	1933	79	40,700	67,000	12,210
Hall-Beckley	0.84	1933	84	93,600	111,000	28,080
Pickens	1.84	1933	92	150,200	82,000	45,060
Snover	0.23	1933	100	16,900	73,000	5,070
Hay	0.20	1933	80	12,600	63,000	3,780
Verdugo	15.24†	1927 and 1933	49	85,000	5,600	25,500
Total				541,500		162,470

* Based on records of Los Angeles County Flood Control District which operates and maintains these basins. † Partly controlled by other basins. ‡ Based on desilting cost of 30 cents per cubic yard.

that produced approximately 659,000 cu yd on January 1, 1934. The difference is probably due in part to the partial restoration by regrowth of cover on the area in the 4-yr intervening period, in part to lack of debris in the channel which had already been cleaned out four years earlier, and to differences in rainfall rates.

The comparison of rainfall for the two storms, as shown in Table 10, may be pertinent. The January 1, 1934, storm lasted approximately 56 hr with 90% of the rain falling in the last 24 hr. The March 2, 1938, storm was broken up into two parts, the first lasting from February 27 to March 1, and the second and more severe part starting on March 2 and in general ending on March 3. The average duration of the first part was 48 hr, and of the last, 32 hr.

Table 10 shows the storm totals of March 2, 1938, to exceed those of the New Year's storm in this region; 52% of the total storm rainfall at the stations listed occurred on March 2. Maximum hourly rainfall amounts varied from 0.63 in. to 1.31 in., this latter amount being nearly the same as the maximum of 1.33 in. in one hour which obtained at the Flintridge Fire Station in 1934.

In general it may be said that, whereas the New Year's storm culminated in an abrupt increase in rainfall intensity producing high peak flows for a short time, which rapidly dropped to negligible flows, the March 2 storm rainfall held up at moderate intensities for a much longer period, producing greater

TABLE 10.—Comparison of Storm Rainfall, 1938*

No.	Station	Feb. 27	Feb. 28	Mar. 1	Sub-total	Mar. 2	Mar. 3	Mar. 4	Sub-total	Grand total	Storm total
251	La Crescenta	1.21	3.35	3.08	7.64	10.27	0.89	....	11.16	18.80	13.50
280	Flintridge Fire Station	1.46	2.71	2.35	6.52	7.77	0.64	....	8.41	14.93	14.92
364	Lower Haines	1.27	2.70	2.36	6.33	7.80	0.85	0.02	8.67	15.00	10.52
360	Haines Debris Basin	1.20	2.58	2.22	6.00	7.47	0.75	....	8.22	14.22	9.37
367	Upper Haines	1.57	3.31	2.95	7.83	10.39	1.16	0.02	11.57	19.40	12.00
373	Briggs Terrace	1.18	3.36	3.15	7.69	9.98	0.86	....	10.84	18.53	12.95
378	La Cañada	1.55	3.20	2.65	7.40	8.99	1.09	....	10.08	17.48	....†
401	Verdugo Mountain	1.42	2.36	2.66	6.44	7.85	0.86	0.02	8.73	15.17	....†
418	Pickens Plots	1.43	3.60	3.14	8.17	8.12	1.06	0.01	9.19	17.36	....†
508	Arroyo Seco Ranger Station	1.26	2.62	2.62	6.50	8.10	1.00	....	9.10	15.60	11.33
577	Los Angeles, U. S. Weather Bureau Station	1.47	2.85	0.08	4.40	6.17	0.49	....	6.66	11.06	8.27
122	Pickens Debris Basin	1.22	2.49	2.64	6.35	8.40	0.57	....	8.99	15.34	....†
647	Tujunga	1.01	2.12	2.07	5.20	6.02	1.88	....	7.90	13.10	11.76
556	La Crescenta	1.18	3.76	3.05	7.99	9.42	0.94	....	10.36	18.35	13.28

* February 27–March 4, 1938, data extracted from Report on Flood of March 2, 1938, by Los Angeles County Flood Control District. Values are not maximum 24-hr rainfall. † Not available—stations started after January 1, 1934.

flow volumes but no peak flows of extremely large proportions in the area under discussion.

The most important comparison of the two floods, and one which shows the actual value of the debris basins, lies in the relative amount of damage caused by water and debris with and without the basins. The flood of January 1, 1934, caused the loss of 34 lives and wrecked 483 houses. It deposited approximately 535,000 cu yd of debris on streets and property. The total damage was estimated at $5,000,000. The debris basins impounded 541,500 cu yd of debris in the 1938 flood which is practically the same volume as was deposited on streets and property in 1934. Almost no damage was done in this region in 1938 and the total cost of desilting the basins was only $162,470, or one thirtieth of the amount of damage in 1934. The right-of-way and construction cost of the nine basins listed, and their concrete outflow channels, did not exceed approximately $3,500,000, which is considerably less than the estimated amount of damage in the one flood of January 1, 1934.

The Los Angeles County Flood Control District has in operation at the present time (1940) a total of sixteen debris basins in the Montrose and other areas. One more basin is under construction and still another will be started soon. They have proved their value in areas where burns have occurred and where debris flows may occur above highly developed residential areas.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

CHANNELIZATION OF MOTOR TRAFFIC

Discussion

By George H. Herrold, Hawley S. Simpson, Arthur G. Straetz, Burton W. Marsh, and Virden A. Rittgers

George H. Herrold,[44] M. Am. Soc. C. E. (by letter).[44a]—The ostensible reduction of distance and the actual removal of traffic hazards by improved transportation facilities creating smooth, safe, uninterrupted traffic flow are the ultimate aims of all traffic studies. Mr. Kelcey has taken another step toward these ultimate goals. One great difficulty in providing roadway facilities for the expanding needs of the motor vehicle is that models of cars can be changed every year or two, but streets and highways cannot be changed so often; they must run their life, some twenty years or more, and while each year's construction is the best that advanced engineering knowledge can provide, the conception, design, and product for any year must be accepted and used for many years.

Fortunately, the improvement in motor vehicles that may be anticipated now will be for more safety and comfort, easier operation, better stopping facilities, etc. Building structural faults out of cars and putting new defensive safety elements into cars have been under way for some time. Probably there will be nothing further in the way of speed, but there may be other developments that the engineer's imagination cannot conceive at the present time. However, it would appear that a chance for the engineer to "catch up" is now before him.

One of the essentials in discussing any subject is nomenclature—a system of words describing the various phases of the art or science whose meaning will not change. It has been necessary, through the years, to adopt sentences and words applying to traffic that the layman could understand; but gradually there have been evolving certain expressions that belong essentially to traffic. Mr. Kelcey has chosen his words carefully. He has used the right word in the right place. In fact, he has evolved a language to express, in workmanlike pictures, the needs of this traffic study.

Note.—This paper by Guy Kelcey, M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Messrs. W. L. Waters, C. J. Tilden, and T. M. Matson; April, 1940, by W. W. Crosby, M. Am. Soc. C. E.; and May, 1940, by Messrs. Julian Montgomery, R. M. Reindollar, Irving Mack, Bruce D Greenshields, T. W. Forbes, and James S. Bixby.

[44] Planning Engr., City Planning Board, St. Paul, Minn.

[44a] Received by the Secretary April 15, 1940.

In 1936 traffic experts were introduced to the friction theory of discussing traffic, medial friction, marginal friction, intersectional friction, and internal stream friction. These words ejected a new line of thinking on the traffic problem and probable solutions, and although the engineer may seldom use these words in his daily conversation, they have been added to his vocabulary, and the theories of friction will thus have their influence on current thinking. An engineer may seldom use his calculus on any given problem, yet the method of thinking engendered by a study of calculus is used by the average engineer continuously, unconsciously. So it will be with the friction theories and also with channelization theories.

Traffic planners have played with a number of curious toys during the years —15 miles per hr on city streets, keeping on the right side of the street to turn left, turning around a center instead of the inside turn, speed zones, U turns, barrier curbs, surmountable curbs, light-reflecting curbs, parallel one-way streets, widening two-lane traffic-ways to four-way traffic lanes, free-ways, the law which gives the right of way to the driver on one's right, many center openings on divided roadways, etc.

Railroad trains travel on rails. Switches, turn-outs, cross-overs and signals are provided for a controlled change of movement. Channelization will bring about this same sure type of control.

Mr. Kelcey's paper is stimulating, and although it propounds new theories for old conceptions, they are sound. He is to be commended for this interpretation.

HAWLEY S. SIMPSON,[45] M. AM. SOC. C. E. (by letter).[45a]—It was not so many years ago that an island in the paved roadway was considered as nothing but an "obstruction" to traffic which no intelligent engineer would condone. There was some slight strain of logic behind such reasoning. Most islands constructed about 1925 to 1930 were located, arbitrarily, at the whim of some well-intentioned citizen more concerned with a vista for a decorative flagpole or World War relic than with efficient traffic movement. Islands thus fell into disrepute and practically all were removed.

Then came the day of wider highways which produced some weird intersection movement possibilities and much resultant property loss and personal suffering. The most forward thinking traffic engineers reasoned that "obstructions" could be put to work if properly placed so as to "obstruct" or "block out" undesirable movements and force desirable ones. Although some argued that this theory was only a return to the flagpole day, it was not long before the "channelization bandwagon" was full to overflowing. Today no street or highway can be called truly modern unless designed around barriers that serve to define channels for proper traffic movement.

This trend is most predominantly noticeable on rural highways, but it also finds acceptance on city streets in the use of center strips, pedestrian islands, and channelization islands. The most practical work in this direction has been done in Milwaukee, Wis.[46] Whether or not a direct result of these activities,

[45] Research Engr., Am. Transit Assoc., New York, N. Y.

[45a] Received by the Secretary April 12, 1940.

[46] "Planning Traffic for Easier Enforcement," by Howard F. Ilgner, *Proceedings*, 6th Annual Meeting, Inst. of Traffic Engrs., 1935, pp. 22–31.

it remains a fact that, for many years, Milwaukee has enjoyed one of the best safety records of major American cities.

Channelization is successful because it is so simple and natural. It guides

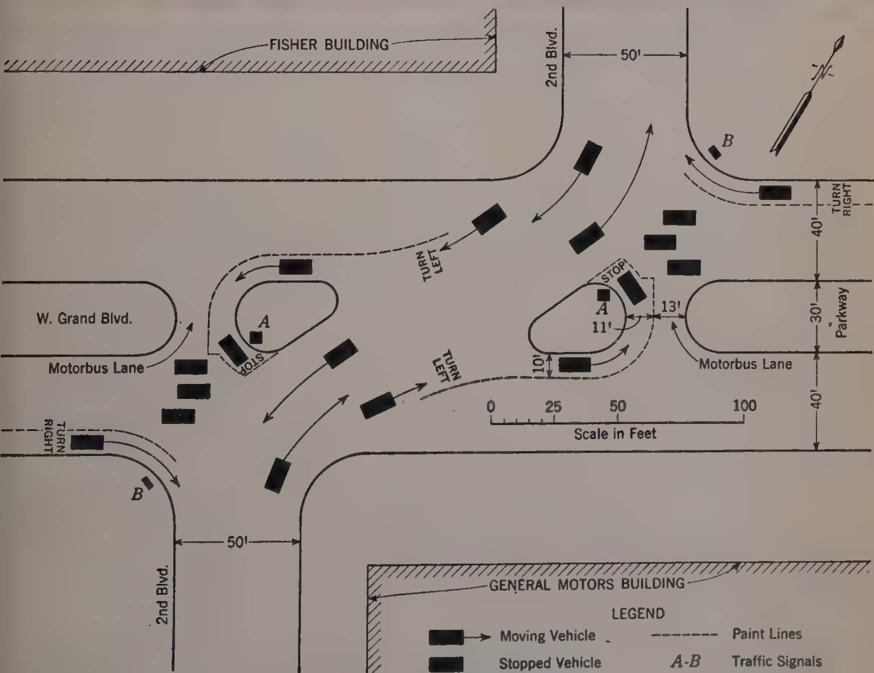


FIG. 53.—CHANNELIZATION OF TRAFFIC AT SECOND BOULEVARD AND WEST GRAND BOULEVARD, DETROIT, MICH.

each motorist through difficult situations, making it impossible for him to do other than the normal and proper thing. Channelization is positive, exact,

and inviolable, easily and readily obeyed by a public which would willingly violate a regulation having the same objective. This is because rules must be over-restrictive to be adapted to the level of the sub-standard driver. Channelization, on the other hand, works equally as well with the sub-standard driver as with the above-par individual. It makes every one a better-than-average driver.

Channelization has many uses. Within the limitations of his paper, Mr. Kelcey could explain only general principles. The writer submits a channelization problem successfully solved in 1925 and functioning as satisfactorily today.

At one of the heaviest traffic intersections in Detroit, Mich. (Second Boulevard and West Grand Boulevard), a substantial volume of left-turning traffic created such congestion that left turns were abolished by order of the police. This created an even worse situation at the adjacent intersections, and so left turns were again permitted—this time around a 6-ft painted traffic circle under the guidance of a police officer. This plan also failed and the writer devised the scheme portrayed in Fig. 53. Straight-through traffic on both Second Boulevard and West Grand Boulevard moves as at any normal intersection. Left-turning vehicles from Second Boulevard, however, take the right-hand lane and turn left around the irregularly-shaped traffic island and there await the green signal on West Grand Boulevard. The left turn from West Grand Boulevard to Second Boulevard is similarly made except that it begins in the left-hand lane. In its simplest terms, this arrangement comprises a traffic circle or oval, split in the center so that north-south traffic moves through, rather than around. It is only one concrete example of how a reduction in the paved area, through construction of a traffic barrier, can be used to promote intersection efficiency and safety.

ARTHUR G. STRAETZ,[47] ESQ. (by letter).[47a]—To find methods for the unimpeded movement of vehicular traffic—that is the gist of traffic engineering and its prime purpose. Continuity of traffic is always coincidental to less friction and less accidents. Focusing a critical eye upon Mr. Kelcey's paper, one must find that this is a very important phase of the art, and that it is well developed by the author.

Fundamentals are difficult to establish and difficult to separate from the maze of accumulated material. Channelization is such a fundamental detail, being almost so simple that its importance is usually overlooked; yet it requires a considerable analysis as well as intense study.

There is a possibility that being too close to the subject, one may develop a mental aberration. One sees too closely and one becomes confused. This is slightly noticeable in the section of the paper entitled "Right of Way." The law—natural and adopted—that "the vehicle from the right has the 'right of way'" is not only an established law in the minds of most drivers but is also a word-play easily remembered.

In the end it is uniform application and enforcement that count most unless the subject is against the grain and foolish.

---

[47] With Kopp Glass, Inc., Chicago, Ill.

[47a] Received by the Secretary April 11, 1940.

As to the technical layout of existing intersections, two methods may be suggested. One is to cover the intersection with a white powder to produce track patterns and to conform the final layout to the trend of least resistance. The other would be to have movable ramps or curbs which could be adjusted easily.

It is interesting to note the tendency of the author to solve most of his problems without the assistance of auxiliaries such as signs, signals, etc. This is significant because it is an attempt to counteract a quite general assumption that signs or signals are the final remedy for the solution of a traffic puzzle. Although the author has interests in mechanical aids to traffic he tries to be logical and succeeds in being so. This is quite in line with the solid professional standard exemplified by the paper, which deserves the attention of any national organization dealing with the traffic problem.

It is quite doubtful whether any other profession as young as that of Traffic Engineering has as complicated and as difficult a task as that of the traffic regulation and the prevention of traffic accidents. He who has been able to keep himself in line despite all the difficulties arising from being on the semi-commercial end of the "game" and yet maintaining a fine professional attitude, as in this paper, is certainly to be commended.

BURTON W. MARSH,[48] M. AM. SOC. C. E. (by letter).[48a]—Not so many years ago, there was extensive opposition among highway engineers to proposals of traffic engineers for the use of islands in roadways. Islands were considered obstructions and no obstruction was viewed as desirable. Fortunately there has been a marked degree of change in such viewpoints, and many, if not most, leaders in highway engineering recognize that various conditions warrant roadway islands. It is becoming generally agreed that warranted islands, properly designed and maintained, benefit motorists and pedestrians.

Mr. Kelcey's splendid paper will do much to clarify thinking on the subject and to encourage greater use and less misuse of such aids to more efficient, convenient, and safe traffic movement.

Greater emphasis seems warranted in forewarning drivers of the presence and location (especially as to physical limits) of islands. This need is especially great at night and under other unfavorable conditions, such as snow, fog, and rain. The better designed and located the island and its "approach end," the less is such need; but the need always exists, for a driver may attempt to overtake and pass a large truck or may undertake some wrong maneuver which he would not make if he had adequate forewarning concerning the island ahead.

Effective lighting of islands is essential, with special attention to approach ends. Such lighting should not only warn the motorist that something unusual is ahead, but should also floodlight the island end so that the driver can see clearly what is ahead in plenty of time to make any necessary adjustment in his path, and to avoid starting any ill-advised maneuver. The higher the speed, the sooner the driver should be guided, because at high speed changes

---

[48] Director, Safety and Traffic Eng. Dept., Am. Automobile Assoc., Washington, D. C.

[48a] Received by the Secretary April 16, 1940.

in lateral position require considerable distance in the direction of travel. Other aids to drivers approaching islands include guide lines on the pavement, gradually rising physical guides of distinctive color and with gradual sloping sides, and island curbs so designed as to indicate their position by light reflected back to the driver's eyes.

Mr. Kelcey is to be commended for not omitting today's most neglected factor in traffic—the pedestrian. All island design should take fully into account the problems, rights, needs, and convenience of the man afoot.

One illustration of the need for more attention to the pedestrian is involved in Mr. Kelcey's discussion under the heading "Driving Factors: Right of Way." Considering vehicles only, Mr. Kelcey states (referring to Fig. 16) that *B*, on his left, is driver *A*'s "first and immediate problem on entering the intersection." A study of 1,008 pedestrian accidents occurring at 76 intersections in Philadelphia (not including turning vehicles) showed that 76% of the pedestrians were struck before they reached the middle of the roadway. Nearly one half (45.9%) were so struck on the near side of the intersection. If, as is customary, the word "intersection" is so defined as to include the cross walks, driver *A*'s first consideration should be for pedestrians.

The aforementioned Philadelphia study also gives support to the present right-hand, right-of-way rule, because the 45.9% situation will be improved if drivers approach intersections slowly enough and attentively enough—in this case, attentive to the right especially—so that they can avoid even much misjudgment and lack of attention on the part of those crossing afoot. Of course, one cannot "look two ways at once."

There is another point favoring the right-hand right-of-way which relates to the vehicular situation entirely. Since it is natural and perhaps instinctive

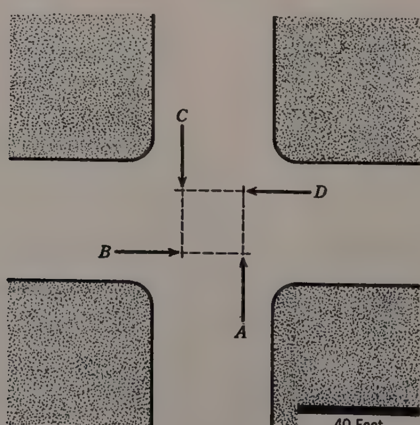


Fig. 54.—Tie-Up Positions of Four Vehicles if There Were a Left-Hand Right-of-Way Rule

for the thinking driver to give attention, as between cross-vehicular-traffic directions, first to the left because that is the cross-pathway which he must meet first, is there not merit in requiring him (by the right-hand, right-of-way

rule) to give proper attention also to the right before he (so quickly) enters the zone of that possible vehicular conflict?

In his Fig. 16, Mr. Kelcey shows how it is possible, with the right-hand, right-of-way rule, for a "tight" tie-up of four vehicles to occur. A companion diagram could be drawn for a similarly possible tie-up of four vehicles if there were a left-hand, right-of-way rule (see Fig. 54). Admittedly, the physical positions of cars would make it easier to clear up the tie-up in the latter case.

VIRDEN A. RITTGERS,[49] ASSOC. M. AM. SOC. C. E. (by letter).[49a]—Channelization, before it can be applied, must be "sold" to the proper officials in many instances. There is one case of a municipality spending thousands of dollars in removing a double-track street-car line and center parkway from a wide major artery and then being urged to paint a 6-in. white center line to keep vehicles on the proper sides of the street. A channelization opportunity was overlooked entirely.

Mr. Kelcey's fine paper will contribute, with certainty, to this problem of selling the principles of channelization. In fact the writer has already found the paper of practical assistance in this respect. Profusely illustrated and written in a style which is impelling through use of clear, concise statements and a splendid choice of words, it is a pleasure to read.

The author deals exclusively with turning movements so far as intersection channelization is concerned. The writer has observed another faulty driving habit which is so pronounced in some urban communities as to be of real import.

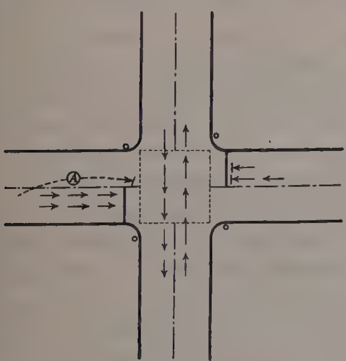


FIG. 55

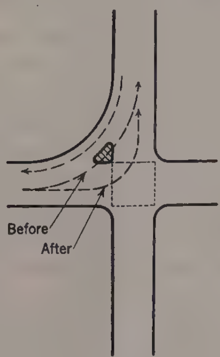


FIG. 56

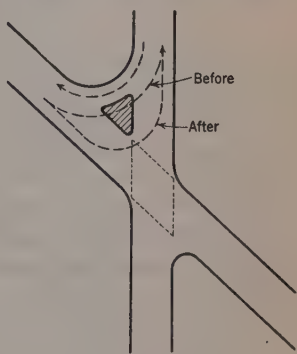


FIG. 57

Fig. 55 illustrates this habit in the approach to a signalized intersection. Driver *A*, in a hurry and not content to take his place behind the vehicles waiting at a red light, pulls to the left side of the roadway and "hogs" his way to the front of the line. Here he constitutes a hazardous interference to cars turning from the cross street and to opposite-direction traffic when the signal changes. Here again channelization with a center island discourages and practically prevents the wrong movement. Seldom will even the unsportsmanlike driver pull to the left half of the roadway when there is a structural defi-

[49] City Traffic Engr., Police Headquarters, San Antonio, Tex.

[49a] Received by the Secretary May 8, 1940.

nition between the left and right sides. Usually the trespassers of this type are "home-town" drivers in a rush to get somewhere during heavy traffic periods. A few drivers will pull over to the left, not knowing the island is ahead, until they have lost their places in line. In such instances they will usually hold up behind the island until they can "horn in" where they belong.

Mr. Kelcey's paper, dealing primarily with the principles of channelization, contains examples of only those intersection approaches which are sufficiently wide to accommodate center islands. Many traffic engineers, when it comes to doing the job, will find so many narrow street complications that it seems worthy of mention. Figs. 56 and 57 illustrate typical urban examples of roadways that are too narrow for center islands but where "corner" channelization may still be applied to regulate faulty left turns (to a substantial extent if not fully) and to facilitate pedestrian crossings and sign and signal placement. Many problems of this type have come about during recent years through the "corner cutting" projects of many cities, particularly under the government relief programs. The writer has in mind one instance in which 40-ft and 50-ft radii were standards on wide residential streets having pavements only 26 to 30 ft wide.

Rows of mushroom-type, concrete traffic buttons have been used in a number of instances to meet the narrow street (and perhaps the cost) problem in channelization. This plan, being a compromise (which is never right) between the feeble paint line and the raised island, is fairly satisfactory for many jobs. Its greatest weakness lies in the difficulty of making the installation visible, thus increasing the "trapping" effect on drivers.

The principles of channelization, as described by Mr. Kelcey, are certainly sound principles. Help for the pedestrian is of major importance (although treated only briefly by the author for reasons which he gives). A subsidiary advantage lies in the increased facility for placing necessary signals and signs at exactly the right spots. This advantage will serve to offset the element of confusion (often more imaginary than real) which might be claimed for some complicated layouts. As a general rule, when traffic is light and speeds higher, the roadway situation ahead is more easily seen and understood; and when traffic is dense, speeds are lower and greater visibility distances are not so essential.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WATER SUPPLY ON UPPER SALT RIVER, ARIZONA

Discussion

BY GEORGE F. MCEWEN, ESQ.

GEORGE F. MCEWEN,[14] ESQ. (by letter).[14a]—A penetrating analysis of available information on stream-gage records and other correlated observations in obtaining the probable future 20-year hydrograph of the Upper Salt River, Arizona, is presented in the paper. Since an inspection of the 46-year hydrograph does not show a tendency even to approximate repetitions of sequences, the author concluded that extrapolation from this short series could not be made. However, he does use past information as a guide for the future by assuming that the observed frequency distribution of nine types of years—flood, very wet, etc.—will prevail in the future. Also, some restrictions as to possible future sequences are provided by preparing duration curves of two consecutive years, and again of three consecutive years, and thus establishing a probable minimum flow in the future for such intervals.

Skilfully, the author takes advantage of certain supplementary data and conditions in this arid region that are especially favorable to long-range forecasting. Although tree-ring measurements are indicators of the amount of precipitation, various other factors often have a great influence, thus obscuring any relation of tree-ring thickness to precipitation. However, in this arid Arizona region enough evidence has been obtained in support of a sufficiently close correlation to make it possible to interpret tree-ring measurements and to determine general trends and cycles in precipitation over hundreds or even thousands of years. From the correlation between precipitation and stream flow the author thus establishes certain important features of the Salt River flow. In particular, severe 5-year droughts such as that of 1900–1905 are thus shown to have occurred in 300-year intervals, beginning with the year 700 A.D. Therefore, the 1900–1905 sequence was neglected in all computations involving

NOTE.—This paper by John Girand, Assoc. M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Dana M. Wood, M. Am. Soc. C. E.; and April, 1940, by LeRoy K. Sherman, M. Am. Soc. C. E.

[14] Prof. of Physical and Dynamical Oceanography, The Scripps Institution of Oceanography, La Jolla, Calif.

[14a] Received by the Secretary April 15, 1940.

the sequence of dry years in estimating probable future sequences of dry years. Mr. Girand has followed the well-established conclusion that runoff data are not entirely fortuitous, and has accordingly realized that their investigation should not be based entirely upon probability methods. In fact, statisticians have long recognized that time series cannot be dealt with by the ordinary statistical methods because the observations ordered in time are in general not mutually independent or random in time. Although the author found no evidence of repetitions of shorter 2-year and 3-year sequences his 5-year means do indicate repetitions—a conclusion that is strengthened by correlations with the sunspot cycles which can be projected more readily. A note[15] confirms the author's 1944 year of increase and 1948 year of sunspot maximum.

In general, it appears that by the skilful use of available information, the author has succeeded in making a forecast of runoff for the 20-year period 1936 to 1956 that should well serve to answer questions as to whether operating requirements of a proposed hydroelectric project would be met.

---

[15] "Forecast Next Sunspot Increase to Begin in 1944, with Maximum in 1948," by J. Q. Stewart and F. C. Eggleston, *Bulletin*, Am. Meteorological Soc., Vol. 21, March, 1940, p. 117.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PRESSURE-MOMENTUM THEORY APPLIED TO THE BROAD-CRESTED WEIR

Discussion

By Messrs. John W. Hackney, Thomas H. Prentice, Boris A. Bakhmeteff, D. D. Curtis, Carl Rohwer, and John Hedberg

John W. Hackney,[23] Jun. Am. Soc. C. E. (by letter).[23a]—In Equation (18) the authors define the relationship between the weir coefficient (Ω), the head on the weir (H), and the height of the weir face (d_4). If, as the authors suggest, K may be assumed to be a constant and equal to 2.00, this formula may be reduced in the following manner:

$$\Omega = \left[\frac{g}{2}\left(\frac{4-1}{4}\right)\left(\frac{d_4 + H}{2\,d_4 + H}\right)\right]^{0.5} = \left[\,12.1\left(\frac{1 + H/d_4}{2 + H/d_4}\right)\right]^{0.5} \ldots (41)$$

In Equation (41), it is apparent that Ω is a function of $\frac{H}{d_4}$ only. Plotting the authors' test results from Table 2 in the form Ω versus $\frac{H}{d_4}$, Fig. 12(a) is obtained. Data obtained from the tests of Bazin, and Cornell University as reported by Robert E. Horton,[24] M. Am. Soc. C. E., have been plotted, in a similar manner, in Fig. 12(b). Also plotted in Fig. 12(b) are the data from the tests of James G. Woodburn,[25] M. Am. Soc. C. E. The curve representing Equation (41) has been plotted on both diagrams. All points plotted are for tests in which the ratio of head (H) to breadth of weir (B) is less than 0.3, to conform to the authors' stipulation that the weirs under consideration are those in which the flow across the weir is nearly parallel to the surface of the weir.

Note.—This paper by H. A. Doeringsfeld, Esq., and C. L. Barker, Assoc. M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Messrs. J. C. Stevens, and H. G. Wilm; and April, 1940, by I. M. Nelidov, Assoc. M. Am. Soc. C. E.

[23] Asst. Hydr. Engr., Aluminum Co. of America, Pittsburgh, Pa.

[23a] Received by the Secretary March 18, 1940.

[24] "Weir Experiments, Coefficients and Formulas," by Robert E. Horton, *Water Supply Paper No. 150*, U. S. Geological Survey.

[25] "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 405.

It appears that in Fig. 12(*a*) the curve representing Equation (41) is a good mean curve for the test points. This is not true, however, in Fig. 12(*b*), in which a mean value for Ω is about constant at 2.6, with an unexplained widening of the band of test points below $\frac{H}{d_4} = 0.3$.

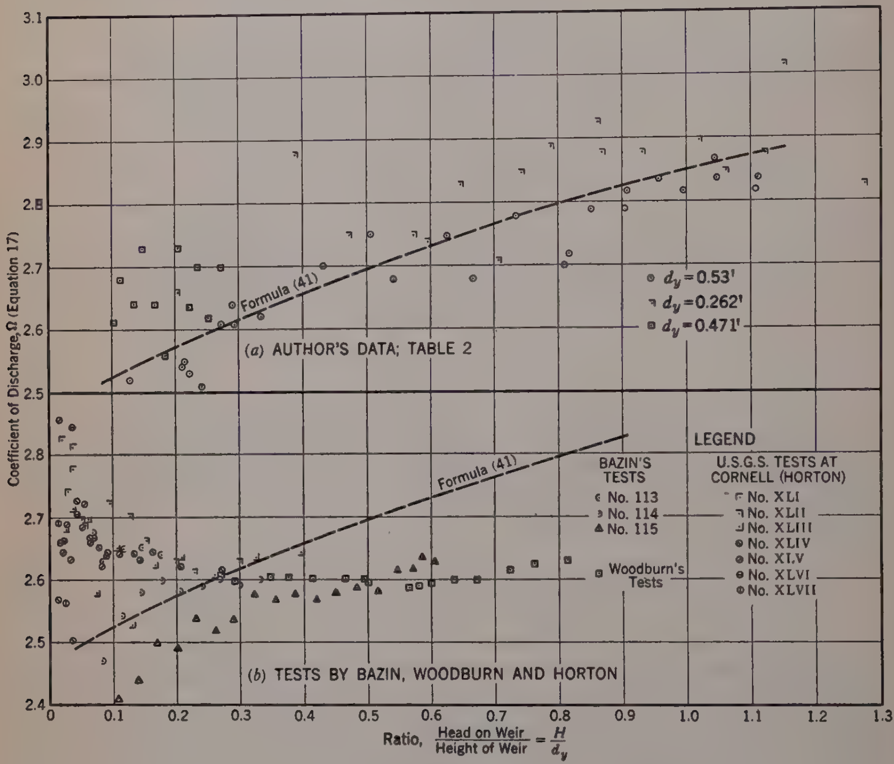


FIG. 12

Figs. 12(*a*) and 12(*b*) concern only tests in which $\frac{H}{B}$ is less than 0.3. Fig. 13 shows the effect on the value of Ω produced by varying $\frac{H}{B}$. This diagram shows the transition zone between the conventional broad-crested weir $\left(\frac{H}{B}\right.$ less than about 0.4$\left.\right)$ and the point where the weir begins to act as if it were sharp-crested $\left(\frac{H}{B} \text{ greater than about } 1.5\right)$.

The authors are to be commended for performing these tests and publishing the results. It is very easy to persuade people to talk and write about test results, but the real need is for people who will take the time and trouble to perform accurate tests.

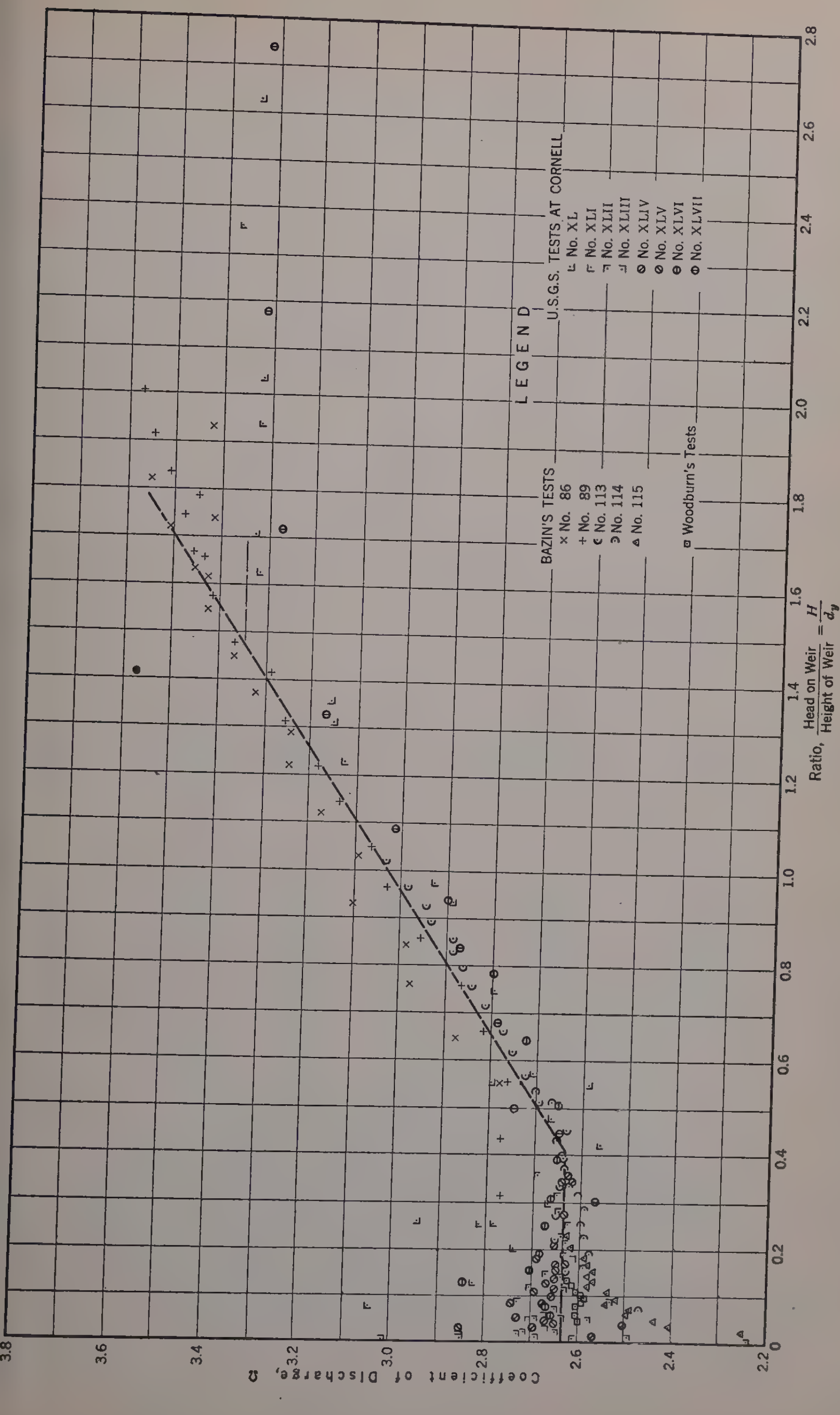


FIG. 13.—DISCHARGE COEFFICIENTS FOR SHARP-CORNERED, BROAD-CRESTED WEIRS AND FOR VARYING VALUES OF $\frac{H}{B}$

THOMAS H. PRENTICE,[26] ASSOC. M. AM. SOC. C. E. (by letter).[26a]—The excellent paper by Professors Doeringsfeld and Barker is a welcomed contribution to the rather scant literature on the broad-crested weir. The discharge formula which they develop, by applying the pressure-momentum theory, is interesting in that it involves the measurement of the two depths: The hydraulic head H, and the depth of minimum, parallel flow.

In advancing their formula, the authors direct particular attention to the necessity of obtaining, accurately, the least flow depth over the weir, since it is at that depth that parallel flow is assumed to exist. From experiments that the writer conducted on broad-crested weirs at Columbia University in 1935, it would appear that there are certain limits beyond which no "minimum" depth will obtain. In fact, from data observed (that is, surface profiles, measured heads, and discharges) it appears that the breadth of the crest L is of great importance in determining the type of flow over the weir. It was

TABLE 5.—RECOMPUTATION OF DATA IN TABLES 1 AND 2

Head, H, in feet	$K = \frac{H}{d_3}$	DISCHARGE, IN CUBIC FEET PER SECOND			Ratio, $\frac{L}{H}$	Head, H, in feet	$K = \frac{H}{d_3}$	DISCHARGE, IN CUBIC FEET PER SECOND			Ratio, $\frac{L}{H}$
		Q (observed)	Ω_0 (observed)*	Ω_c (computed)†				Q (observed)	Ω_0 (observed)*	Ω_c (computed)†	
(1)	(2)	(3)	(4)	(5)	(6)	(1)	(2)	(3)	(4)	(5)	(6)
(a) MINNESOTA TESTS (SEE TABLE 1(a))											
0.117	1.95	0.170	2.53	2.59	19.2	0.355	1.97	0.955	2.69	2.76	6.3
0.128	1.83	0.193	2.50	2.63	17.6	0.430	1.96	1.280	2.70	2.81	5.2
0.156	1.84	0.270	2.61	2.66	14.4	0.480	1.92	1.560	2.79	2.85	4.7
0.230	1.92	0.500	2.70	2.69	9.8	0.528	1.88	1.820	2.82	2.89	4.3
0.270	1.93	0.647	2.75	2.72	8.3	0.552	1.94	1.973	2.86	2.88	4.1
(b) WASHINGTON TESTS (SEE TABLE 1(b), HEIGHT, 0.262 FT)											
0.053	1.29	0.027	2.66	2.39	21.1	0.207	1.83	0.227	2.89	2.85	5.4
0.090	1.49	0.065	2.89	2.68	12.4	0.226	1.86	0.262	2.93	2.86	5.0
0.103	1.64	0.079	2.87	2.73	10.9	0.228	1.90	0.2615	2.88	2.85	4.9
0.124	1.70	0.100	2.75	2.76	9.0	0.245	1.90	0.291	2.88	2.87	4.6
0.132	1.86	0.110	2.75	2.74	8.5	0.268	1.88	0.335	2.90	2.90	4.2
0.151	1.92	0.1348	2.76	2.75	7.4	0.279	1.86	0.350	2.85	2.92	4.0
0.157	1.84	0.142	2.74	2.78	7.1	0.295	1.88	0.385	2.88	2.94	3.8
0.170	1.77	0.165	2.83	2.82	6.6	0.302	1.90	0.418	3.02	2.92	3.7
0.186	1.86	0.181	2.71	2.82	6.0	0.334	1.90	0.455	2.83	2.95	3.3
0.195	1.82	0.204	2.84	2.84	5.7	....	....	....	....	....	....

* From Equation (42). † From Equation (18).

found that if the breadth of crest L and the hydraulic head H are related by writing the ratio $\frac{L}{H}$, flow approximating the parallel pattern exists only when that ratio is greater than 5. For values less than this the flow was distinctly curvilinear over the entire breadth of the crest. Therefore, the proper appli-

[26] Asst. Prof., School of Technology, Coll. of the City of New York, New York, N. Y.

[26a] Received by the Secretary April 3, 1940.

cation of the pressure-momentum theory should be in those cases in which $\frac{L}{H}$ has a value greater than 5.

Another assumption of the pressure-momentum theory is that the pressure distribution on the upstream contracted section is hydrostatic. That observation is an approximation, permissible only within certain limits of $\frac{H}{d_4}$, in which d_4 is the height of the weir. At the upstream entrance to the weir, the manometers which the writer used to record pressures on the weir crest indicated in a very striking manner that there was a region of under-pressure, at the entrance, by a very decided dip in the pressure profile. In this region the flow is most sharply curvilinear, the surface profile being convex upward. Pressure profiles plotted above the crest showed other regions of curved flow, being above the surface profile when the flow was concave upward (pressures greater than hydrostatic) and below the surface profile when the flow was convex upward (pressures less than hydrostatic).

Manometer readings for the piezometer connections on the vertical upstream face of the weirs showed that the pressures were substantially hydrostatic for the lower points on the face, but that there was considerable deviation of the pressure line inward toward the weir face in the region adjacent to the crest level; such departure from the true hydrostatic pressure was due to the velocity head of approach of the oncoming flow.

TABLE 6.—VALUES OF C, COLUMBIA TESTS; $d_4 = 0.667$ FT AND $b = 0.50$ FT

Head, H, in feet	Observed discharge, Q, in cubic feet per second (1)	Coefficient, C, in Equation (44) (observed) (2)	Ratio, $\frac{L}{H}$ (3)	Observed discharge, Q, in cubic feet per second (1)	Coefficient, C, in Equation (44) (observed) (2)	Ratio, $\frac{L}{H}$ (3)
	(a) LENGTH OF WEIR $L = 2.00$ FT			(b) LENGTH OF WEIR $L = 3.00$ FT		
0.100	0.086	2.70	20.0	....	....	....
0.200	0.243	2.69	10.0	0.244	2.70	15.0
0.300	0.449	2.69	6.67	0.452	2.71	10.0
0.400	0.698	2.70	5.00	0.703	2.72	7.5
0.500	0.986	2.71	4.00	0.995	2.73	6.0
0.600	1.312	2.72	3.33	1.325	2.75	5.0

In advancing new formulas resulting from tests, the question is generally raised as to whether there is a simplification and advantage over the formulas already in general use. Being interested in comparing the results of his tests with those obtained by the authors, the writer recomputed some of the data appearing in that paper, as shown in Table 5. The values of Ω in Columns (5) are discharge coefficients computed from Equation (18). The values marked as observed are determined from

$$Q = \Omega_0 \, b \, H^{1.5} \dots\dots\dots\dots (42)$$

Both sets of values were based upon the values of the discharge, Q, given in the authors' data. There is also presented in Table 6 values of the respective

discharge coefficient, C, using Francis' formula for data obtained in the Columbia tests:

$$Q = M \times \tfrac{2}{3} \times b \times \sqrt{2\,g}\left[\left(H + \frac{v^2}{2\,g}\right)^{1.5} - \left(\frac{v^2}{2\,g}\right)^{1.5}\right] \ldots\ldots (43)$$

in which

$$M \times \tfrac{2}{3} \times \sqrt{2\,g} = C \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

It will be observed that, in general, the value of C in Table 6 is rather steady, approximating the commonly used average, C = 2.70 (see Fig. 14).

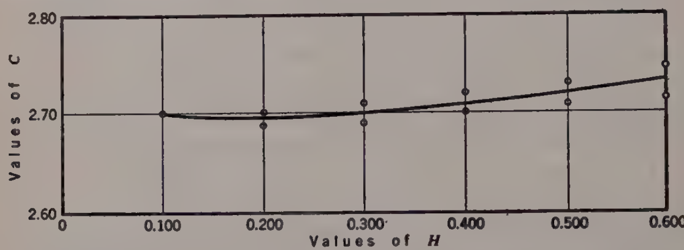


FIG. 14.—PLOTTED VALUES OF C VERSUS H IN FRANCIS' FORMULA, EQUATION (43)

The writer is in accord with the statement made by the authors that there should be considerable caution in applying their formula in cases where there is curvilinear flow over the entire breadth of the crest. Might not the question be asked whether some of the discrepancies in the values of Ω in the formula offered by Professors Doeringsfeld and Barker (Equation 18) could not be caused by difficulties in obtaining true minimum depths because of sharply curvilinear flow?

BORIS A. BAKHMETEFF,[27] M. AM. SOC. C. E. (by letter).[27a]—An interesting attempt to apply the momentum principle in establishing the interrelations which govern the inflow to a broad-crested weir is presented in this paper. The writer presumes that the analysis is meant to apply to a sharp-edged entrance only, for in this case the vein jumps the crest and produces a roller such as that in Fig. 15. "Separation" and the brusque change of flow forms

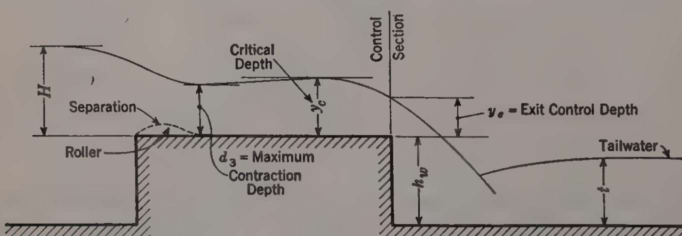


FIG. 15

are accompanied by a substantial local entrance loss. The mechanical premises resemble those in a hydraulic jump or in the sudden expansion of a closed conduit. The tangential friction plays a negligible rôle and can be omitted from

[27] Prof., Civ. Eng., Columbia Univ., New York, N. Y.

[27a] Received by the Secretary April 13, 1940.

consideration, while the intense energy losses, due to concentrated local turbulence, are accounted for by the pressure differences in the controlling sections. Experience shows that, notwithstanding the approximate character of the analysis, the momentum principle in such cases expresses the bulk interrelations in quite a satisfactory manner. The instance of the sharp-edged, broad-crested weir, investigated by the authors, offers one more proof of the accord between theory and experiment. K. Woycicki applied[28] the momentum principle under very similar conditions to investigate the roller-covered submerged efflux from under a sluice.

The authors' findings center around Equation (18), which constitutes a "discharge coefficient," the numerical value of which can be determined by inserting into the formula the two easily measurable quantities, H and d_3. It appears that in this manner Equation (18) can be applied usefully for approximate bulk appraisals, such as in the design of structures. On the other hand, neither Equation (18), nor any other customary formula derived from the momentum principle, possesses the degree of accuracy required for precise metering.

It is questionable whether the very thought of using the entrance section of the broad-crested weir as a metering device is expedient, in general, particularly since Hunter Rouse,[29] Assoc. M. Am. Soc. C. E., disclosed the advantages of the exit section as a possible control, pointing out that, for fully aerated nappes, the ratio $\frac{y_e}{y_c}$ (Fig. 15) remains consistently stable and equal to 0.715. The possible disadvantage in Mr. Rouse's method is the difficulty of providing adequate ventilation for the nappe under field conditions. Therefore, further experiments were conducted by T. B. Robinson, Jun. Am. Soc. C. E., in the Fluid Mechanics Laboratory at Columbia University with the view of establishing whether it was practicable to use the exit section of a broad-crested weir as a control in the presence of unventilated nappes. The results obtained indicate that, notwithstanding the wide change in tailwater depth $\left(\frac{t}{h_w}\text{ up to near } 0.9\right)$, the ratio $\frac{y_e}{y_c}$ for non-aerated nappes remained remarkably steady, the average value of $\frac{y_e}{y_c} = 0.665$ representing observations with an accuracy within 1%.

D. D. Curtis,[30] Assoc. M. Am. Soc. C. E. (by letter).[30a]—Both in its presentation of new experimental data and in its use of the momentum theorem, this paper furnishes an addition to the literature on the subject of broad-crested weirs. Any application of the momentum principle is interesting. Because of the mutuality, with the resulting equational cancellation, of forces involved, the principle commonly known as the conservation of momentum is

[28] "Wassersprung, Deckwalze und Ausfluss unter einer Schutze," by K. Woycicki, *Verlag* der Polnischen Akademie der Technischen Wissenschaften, Warsaw, 1931.

[29] "Discharge Characteristics of the Free Overfall," by Hunter Rouse, *Civil Engineering*, April, 1936, p. 257.

[30] Prof. of Mechanics and Hydraulics, Clemson Coll., Clemson, S. C.

[30a] Received by the Secretary April 15, 1940.

one of the neatest devices of mechanics. However, as in most such applications in hydraulics, the problem of the paper is one of continuous action of forces on a body with consequent change in momentum, rather than of the interaction of two bodies on each other, as implied in the momentum-conservation principle.

In cases where conservation of momentum does not apply, full consideration of all forces involved is essential to rigidity. Some years ago, in conducting research on the expanding stream below a particular submerged weir, the writer had occasion to apply the momentum formula in predicting water-surface elevations. The results were gratifyingly successful, although it was not found possible to consider certain of the less important forces.

It is evident that the authors ignored certain items in addition to the mentioned friction loss between the crest and the section of parallel flow. For example, there is a downward acceleration of part of the water. This can be produced only by a reduction of the pressure intensity upward on the water to a value less than the static pressure since there must be a resultant force downward, and the only downward force acting on the water—the gravity pull—is the same in amount as the static pressure. The effect on the derivation of neglecting this pressure reduction is problematical, but it seems likely to influence the assumption of hydrostatic pressure on the upstream face of the weir. Furthermore, the momentum (which is scarcely to be considered a force, but rather a mass-velocity property) is not correctly computed from the average velocity. It will be noted that the error from the use of average velocity for calculating momentum is of the same order as that from using average velocity in computing velocity head. This is one degree less in the velocity exponent than that in computing kinetic energy on the same assumption.

In regard to this latter point (the effect of use of average velocity) it is possible to see something of the magnitude of the error in the instance of momentum (and velocity head). With uniform velocity variation from zero to V between the vertical limits of the stream, the true momentum would be twice that calculated on the basis of the average velocity. For parabolic distribution from zero to V, the parabola being concave to the axis from which velocity is measured, the true momentum would be 1.35 times the calculated one; with uniform variation from 0.5 V to V the true momentum would be 1.11 times the calculated one; with uniform variation from 0.67 V to V the true momentum would be 1.04 times the computed one. This can be seen to hold possibilities of serious error, dependent upon the velocity distribution. In the case of the theory developed in this paper, it is not likely to be significant.

To afford a comparison with the authors' results, data were recomputed for runs made by the writer on a weir 1.5 ft high, with crest rounded to a 9-in. radius, with upstream face sloped at 40° and tangent to the rounded crest, and with downstream face similarly tangent to the crest circle but adjustable for slope. The main purpose was quite different from that of this paper, and conditions under which the weir was operated in many runs were not suitable to direct comparison with the authors' work. For such as were made under a comparable regimen of flow, values of K in Equation (16) were found to

range close to 1.90. The fact that the value of K obtained under decidedly different conditions closely approached that found by the authors gives weight to the belief that the items disregarded are unimportant in their effect. In any event the size of Ω is not particularly sensitive to the value of K. Calculations show that reducing K from 2 to 1.97 would make a difference of less than 1% in Ω; reduction to 1.94 would correspondingly make a difference of less than 2%; and further reduction to 1.90 would make less than 2.5% difference from that obtained for $K = 2$. Therefore, the use of the value 2 appears warranted.

In spite of the defects pointed out, the paper is interesting as a new approach to the problem of the broad-crested weir. It is hoped that the work will be continued and expanded. For instance, rounding of the upstream edge of the crest would make for greater stability in the water surface with probably even closer agreement of results with theory.

CARL ROHWER,[31] Assoc. M. Am. Soc. C. E. (by letter).[31a]—The possibilities of broad-crested weirs as measuring devices have long been recognized by engineers, and extensive tests have been made to determine their characteristics. It was assumed that by measuring the critical depth the discharge over this weir could be determined easily and accurately because of the simple relation existing between the critical depth and the discharge.

A series of tests by James G. Woodburn,[3] M. Am. Soc. C. E., showed, however, that the location of the critical depth varied with the shape of the weir and with the head, and that consequently the critical depth could not be used as the basis in determining the discharge. R. L. Parshall, Assoc. M. Am. Soc. C. E., conducted tests on several types of broad-crested weirs and found similar results.[32]

The application of the pressure-momentum theory to the flow over broad-crested weirs is another attempt to derive a formula suitable for measuring the flow over this type of weir. The formula derived by the author (Equation (16)) is not so simple as the critical-depth formula, but since it involves all the factors influencing the flow it gives a direct solution of the discharge. In the formula, K is substituted for the ratio of the head H, on the weir, to the depth d_3, on the top of the weir in the zone of parallel flow. If K is known, then the measurement of the single head H determines the discharge. Since the tests made by the authors show that K is not constant (Table 1, Column (3)), it seems to the writer that it would be desirable to make direct measurements of both H and d_3, thereby eliminating the uncertainty as to the value of K. It is true, as stated by the authors, that small variations in K do not seriously affect the results (Table 2, Columns (5), (6), (11), and (12)), but, in case the tailwater were high enough to retard the flow over the weir, there would be a material change in K and probably also in the discharge computed

[31] Irrig. Engr., Div. of Irrig., SCS, U. S. Dept. of Agriculture, Colorado Experiment Station, Fort Collins, Colo.

[31a] Received by the Secretary April 5, 1940.

[3] "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 387.

[32] Discussion by R. L. Parshall of "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 440.

by the formula unless the true value of K was used. Since the derivation of Equation (16) is a general one (in that the degree of submergence is not a factor in the accuracy of the analysis by the pressure-momentum theory), it seems to the writer that this is an important feature of the formula and that tests should be conducted to find the degree of submergence permissible without affecting the results.

The computed curves and the experimental results shown in Fig. 9, for weirs with heights of 1.424, 1.256, 0.965, 0.781, and 0.313 ft, indicate that, with a few exceptions, the observed discharges are consistently greater than the computed discharges. This condition might result from an error caused by the method of measuring the gage heights or from some error in the assumptions which have to be made in applying the pressure-momentum theory to the problem of flow over broad-crested weirs. Whatever the cause of the discrepancy in the results, the differences are so small that a slight correction of the formula in the nature of an empirical constant or coefficient would apparently bring the observed and computed results into very close agreement.

The profiles of the water surfaces and the pressure heads in Fig. 10 show that piezometer readings may differ materially from the true water depths. In fact, the differences in zones where the water-surface profiles have a pronounced curvature are so great that piezometer readings would give an entirely erroneous indication of the true water depth. Where parallel flow exists, the water surface and the piezometer surface are nearly the same. It is interesting to note that Professor Bakhmeteff in his discussion of Professor Woodburn's paper[15] demonstrated that when the surface of the water was convex the piezometric surface would be below the water surface and that when the water surface was concave the piezometric surface would be above the water surface. This is shown clearly in Fig. 10. At points of inflection in the water-surface curve the water is not changing direction and consequently the two surfaces should agree. This fact also is confirmed by the profiles in Fig. 10.

The broad-crested weir cannot take the place of the sharp-crested weir in the laboratory, and in the field it has some of the same limitations that restrict the use of the sharp-crested weir. The principal objection is that the pool formed in front of the weir is a natural settling basin for debris. The accumulation of material in the pool reduces the height of the weir and also increases the velocity of approach; consequently the formulas generally applied give erroneous results. The formula derived by Messrs. Doeringsfeld and Barker for broad-crested weirs is subject to the same criticism because the height of the weir is one of the factors in the formula, and obviously if the height has been reduced by the accumulation of silt, the formula will not give correct results unless the actual height of the weir is determined each time before applying the formula. It is believed by the writer that control sections, such as the Parshall flume,[33] are more satisfactory than weirs for use in streams and canals because there is no obstruction in the bottom of the channel and consequently no tendency for silt to deposit under ordinary conditions.

[15] Discussion by Boris A. Bakhmeteff of "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), pp. 423–434.

[33] "The Improved Venturi Flume," by R. L. Parshall, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), pp. 841–880.

JOHN HEDBERG,[34] ASSOC. M. AM. SOC. C. E. (by letter).[34a]—On a slope steep enough to warrant the use of the cosine of the angle of inclination the pressure is definitely not equal to the weight of the column of liquid above it. A more exact value is considerably less than this.[35] However, the entrainment of air that usually accompanies steep-slope flow makes each of the terms of equation (9) too uncertain for use. The fact that the authors apply their theory only to the special case of horizontal flow makes everything come out all right so that the final Equation (16) is correct.

The authors should present the pressures on the downstream face of the weir and extend the surface profile some distance downstream. These data will be very valuable for future investigations.

Another point that deserves attention is the rôle of the hanging eddy along the upstream edge. The authors are correct in saying that curvature of flow makes the simple critical-depth formula inaccurate, but they failed to note that the eddy hanging along the sharp crest acts like an additional height of weir. The flow in the central portion of the weir is made subcritical by the flow down the sloping rear face of the eddy region. Possibly the fluctuations in the size of this vortex space are the major factors in the variations in the value of K. Further study of this point would seem desirable.

Corrections for *Transactions:* December, 1939, *Proceedings*, page 1721, Equation (9), change "tan θ" to "sin θ"; line following Equation (12), change "force" to "function"; Equation (3), change the left-hand side to read "$[P_1 \Delta A_1 - P_2 \Delta A_2 + \frac{1}{2}(\Delta A_1 + \Delta A_2)\Delta l' \sin \theta]t$"; page 1722, line 10 from the bottom, change "breadth" to "length" and "length" to "breadth "; in Fig. 9 change "p" to "d_4" in each sub-caption; page 1729, line 2, change "1.98" to "1.93"; in Table 1, last column, the first five items should be marked "standing waves on weir"; page 1729, line 10 from the bottom, change "Equation (10)" to "Equation (16)"; Table 3, headings of Columns (3), change "Equation (17)" to "Equation (18)"; page 1731, in definition for C_1, change "$L\ H^{1.5}$" to "$\frac{Q}{L\ H^{1.5}}$"; in March, 1940, *Proceedings*, page 563, change line 9 from the bottom to read "* * * is uniform and equal to V_1, the dynamic pressure is $\frac{w\,b\,d_4}{g} V_1(1 - \cos \alpha)$ where α * * * "; page 565, line 3, change "α" to "a"; see also corrections for *Transactions* on page 570.

[34] Asst. Prof., Civ. Eng., Stanford Univ., Stanford University, Calif.

[34a] Received by the Secretary March 22, 1940.

[35] "Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, Assoc. M. Am. Soc. C. E., McGraw-Hill Book Company, Inc., New York and London, 1st Ed., 1938, p. 281.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ANALYSIS OF LEGAL CONCEPTS OF SUBFLOW AND PERCOLATING WATERS

Discussion

By Harold Conkling, M. Am. Soc. C. E.

Harold Conkling,[81] M. Am. Soc. C. E. (by letter).[81a]—This discussion relative to the subject matter of the author's excellent paper is limited mainly to a typical stream, in an alluvial filled valley, flowing underground or partly on the surface in its more normal regimen. It does not concern itself with the phenomena associated with the occasional large flood which may course through a surface channel so rapidly that there is not time for conditions to adjust themselves.

Study of ground water makes revision of the definition of such a stream desirable. A former concept is expressed in the quotation which the author cites (see heading "Subsurface Water Course"): "A 'stream of water' has a defined channel. It has banks, and is very distinct from the percolations of subsurface water, which oozes in veins or filters through the earth's strata."

This definition describes only the surface portion of the stream. The entire stream may be many times wider and deeper and may include "the percolations of subsurface water." It may extend downward and laterally into the alluvium. It may have a bank or banks, but not necessarily so. If so, the bank may be a sudden and definite change from permeable alluvium to material so fine as to be practically impermeable; or the bank may be the bedrock or other material that bounds the valley and rises as hills on either side. The boundary of the stream may be indicated only by a radical change in direction of flow of the underground water, and in such case it is not definite. Neither is a definite boundary readily discernible when two bodies of water flowing on the surface meet unless the waters are of different color. Although it may not be definite, yet if information is sufficient, it will be evident that the boundary is within a zone which can be delimited and this is true whether the water is on the surface or underground.

Note.—This paper by C. F. Tolman and Amy C. Stipp was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1940, by Donald M. Baker, M. Am. Soc. C. E.; April, 1940, by Messrs. Samuel C. Wiel, Hyde Forbes, and Ronald B. Harris; and May, 1940, by Messrs. Edward F. Treadwell, O. E. Meinzer, M. R. Lewis, and Bayard F. Snow.

[81] Deputy State Engr., Water Rights, Sacramento, Calif.

[81a] Received by the Secretary April 15, 1940.

The water in the surface manifestation of a stream moves at varying speeds. At one point it may be stationary and nearby in rapid movement. Generally it is quietest near its banks. Its particles move vertically and laterally with reference to the general body of the water. Any particle may be above ground here and below ground there, or the reverse. The dominating phenomenon is that the particles move as a whole in the same general direction, although the direction of part of them may be reversed for a short time. A study of the movement of surface streams will show some of the many variations.

The following definition of a stream is offered: "A stream is a body of water moving as a whole in an accustomed location and in a definite direction." This defines the Gulf Stream, or any ocean current, as well as a land stream, and the definition must be thus broadened because a fresh-water stream in alluvium is often (in fact, generally) a body of water moving over and through alluvium filled with water. The portion of the surface manifestation of the stream which flows the fastest is water moving through water and, in spite of the interference caused by the particles of alluvium, the underground portion is water moving through water just as the Gulf Stream is a body of water moving through water.

This conception and definition does not agree with Figs. 9 and 10 and the text in Part I, under the heading "Subflow Indicated by Contour Pattern." As indicated by the water-table contours, the boundary of the stream is not the dashed line shown in the figures but some other more distant line.

The quotation cited by the author to which reference has heretofore been made goes on to state that "An 'underground stream' of water differs from a surface stream only with respect to its location above or below the surface." This is correct if the concept of the surface stream is as the writer has outlined in the foregoing paragraphs. However, since the concept of a surface stream in the mind of the author of the quotation is faulty, his concept that water moving through alluvium beneath the surface is necessarily not a stream is also faulty. With reference to a line at right angles to the axis of direction of movement, a so-called surface stream may be (and generally is) partly underground; an underground stream is wholly underground. Again, with reference to a similar line, the underground stream may be partly a surface stream a few rods either upstream or downstream from the observer or the partly surface stream may be an underground stream.

In the typical underground basin of the mountainous and more or less arid region of the United States, the underground water may have its source in several streams which enter the basin, and each one of these may produce a subsurface stream which coalesces with the others to form a main stream that moves to the basin outlet. In other words they are tributaries. Water is a comparatively nonviscous fluid which moves rapidly with the various forces acting upon it; but underground where it is capable of moving only through the small interstices of the alluvium which impedes it, it acts as a highly viscous fluid, and its rate of movement as compared to the rate of movement of water on the surface may be almost infinitely slow. Therefore, as compared to a so-called surface stream carrying the same amount of water, the underground

stream may be very wide and deep and the water table may have a much greater slope than the surface portion of a stream.

The conclusion from the foregoing is this: Generally, conflicts over use of ground water arise only after its use has become extensive, and consequently there probably exists a considerable body of information from wells and surface indications as to its movement. Except for a slight technical difference that it is unnecessary to discuss herein, ground water moves under the influence of gravity as surface water does, except that its rate of movement is much slower and the phenomena of rapid movement are absent. Since the phenomena associated with the movement of surface water are well known, trustworthy deductions can be drawn in many cases as to the movement of underground water even if all information concerning it is necessarily derived from measurements at wells defining the water table at those points.

With the information now available, and its crystallization into the conception of a stream hereinbefore expressed, it would seem probable that, in the future, the subsurface water involved in many cases can be treated as a stream by experts and courts, thus removing the concepts in previous decisions which the author mentions.

As a further matter to consider, it may be that hydrologists, geologists, and engineers have used a different nomenclature too freely in describing underground water and that, with the accompanying and necessary definitions, confusion instead of clarification has resulted. Although it may be acted on by different forces in different locations, water is water whether on or below the earth's surface and the phenomena associated with the movement of surface water are part of the experience of most men.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

MINIATURE SYSTEM OF FIRST-ORDER ALINEMENT AND TRIANGULATION CONTROL

Discussion

By Messrs. William Bowie, Earl O. Heaton, George H. Dell, Charles B. Stanton, and C. L. Garner

William Bowie,[5] M. Am. Soc. C. E. (by letter).[5a]—No doubt methods similar to those used by Mr. Hough in making an alinement survey and a triangulation at Tygart Dam, as a means by which horizontal movements of the land near the dam and distortions of the dam itself can be detected, will be used at others of the many dams that have been erected in recent years. That knowledge of the degree of stability of a dam structure and of the surrounding area is desirable admits of no doubt. The problem confronting engineers has been the devising of a method by which this could be done. Mr. Hough has solved the problem successfully.

The apparatus used by him in the alinement survey is ingenious; of particular interest is the lamp described. Engineers wishing to conduct alinement surveys and triangulation should profit by Mr. Hough's experience and make their observations at night. The officials of the U. S. Coast and Geodetic Survey also learned that observations for triangulation could be made more rapidly, accurately, and economically at night than during daylight.

In order to do good work on such a project as that which Mr. Hough describes, the best of instruments should be used. It is wasteful to use transits and poor direction theodolites since finer instruments are now on the market and can be purchased at reasonable costs. Engineers who are satisfied with only the most up-to-date machines and other equipment seem to be loath to obtain high-grade surveying instruments and retire the old-fashioned ones.

The exceedingly high accuracy secured by Mr. Hough in his triangulation is noteworthy, especially since his lines were short. One familiar with the textbooks on higher surveying of the past generation will recall that primary triangulation (now designated first-order) required triangle sides that were 10

Note.—This paper by Floyd W. Hough, M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[5] Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey (*Retired*), Washington, D. C.

[5a] Received by the Secretary March 4, 1940.

miles or more in length. In fact, triangulation of primary, secondary, and tertiary character referred more to the lengths of lines than to accuracy.

For years the officials of the U. S. Coast and Geodetic Survey have advocated the use of geodetic methods and instruments on even comparatively small survey jobs. Any engineer who really knows a surveyor's transit and the micrometer microscope used in measurements in a physics laboratory can obtain acceptable results, after a few hours' practice, with a good direction theodolite. With correct eyesight and more experience he can obtain excellent results.

Geodetic surveying such as Mr. Hough describes is high-class engineering. What he accomplished is a challenge to other engineers which it is hoped they will accept.

Earl O. Heaton,[6] M. Am. Soc. C. E. (by letter).[6a]—It was gratifying to read Mr. Hough's excellent paper and to note at what a small percentage of the cost of the construction project it was completed. In demonstrating how small the cost of an accurate survey really is, this paper should be helpful in persuading other construction executives, in public and private enterprise, to execute surveys of this type.

A point which Mr. Hough did not mention, but which seems very important, is the possibility of the future use of this miniature system of control for expansion of the present project, or for other new engineering and construction projects in the immediate vicinity. Mapping and surveying of any nature that might seem desirable can then be started from this existing miniature system.

Instruments and methods for the triangulation described by Mr. Hough are similar to those which were used by the writer on first-order triangulation and traverse control surveys of Rochester, N.Y., and Atlanta, Ga. On these projects it was also found that short lines do not make accurate observations difficult, provided reasonable care is used in centering lights and instruments.

In making the aforementioned surveys, small lamps similar to those described by Mr. Hough were used on traverse lines much less than 1,000 ft long. Although the lamp slit was much wider than $\frac{1}{8}$ in., the additional width of opening did not affect the accuracy of the observations adversely. Great care was exercised, however, to eliminate possible eccentricity in the reflector and bulb, and this was accomplished by placing a piece of thin paper or ground glass over the slit.

George H. Dell,[7] Assoc. M. Am. Soc. C. E. (by letter).[7a]—The application of modern first-order methods to the establishment of precise reference lines for the study of horizontal displacements in large structures is ably described in this paper by Mr. Hough. The process involves two major steps of equal importance—namely, monumentation and observational procedure.

It is evident that the monumentation system of the Tygart Dam alinement has been carefully planned so as to permit the lines to be re-established at

[6] Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C.

[6a] Received by the Secretary March 11, 1940.

[7] Associate in Civ. Eng., Univ. of Illinois, Urbana, Ill.

[7a] Received by the Secretary April 22, 1940.

any future time even if some of the monuments become disturbed. Each of the two alinement lines extends over a considerable distance and is permanently monumented at six points, the monuments being supplemented by transverse reference monuments. The triangulation system, the lines of which intersect the alinement lines at varying angles, serves to provide a means of determining the directions and magnitudes of probable earth movements at various points. The fact that the dam design was such as to require a separate line for the spillway alinement has apparently resulted in a strengthening of the system. Since the two lines were established independently, the monuments which were installed at a number of opposite points serve not only as transverse reference marks for the detection of future disturbances but provide additional checks on the original alinement observations. This feature of double alinement would appear to be desirable in any case. In this connection, it occurred to the writer that it might have been advantageous to have alinement bars installed on the spillway line opposite those placed on the main alinement. It is probable that such a system would provide additional aids in detecting alinement errors due to possible disturbances to the monuments and in determining differential displacements of points on the monoliths.

The paper treats the details of observational procedure with due consideration to their importance, and it should serve as a valuable guide to engineers who are called upon to execute similar projects.

At the same time, engineers who are faced with the problem of fixing relatively accurate alinements with limited instrumental facilities will find many useful suggestions in the paper, such as having the instrument in good adjustment; shielding the instrument and plumb-bob from the effects of wind; centering over the points with care; leveling the instrument with the aid of the telescope bubble; setting points on line, when possible, by sighting on a distant foresight; using a shuffleboard or a suitable substitute for obtaining the readings; and guarding against phase errors. Points which have to be set by plunging should later be rectified by a series of direct observations. It is also important that the bearings which support the horizontal axis be free from looseness or irregularities due to wear.

The exact sequence that was followed in establishing the points on the Tygart Dam alinement does not seem to be entirely clear; it would be helpful if the author would elaborate on this phase of the problem in his closing discussion.

Charles B. Stanton,[8] M. Am. Soc. C. E. (by letter).[8a]—In his excellent, although somewhat brief, paper Mr. Hough has presented a simple procedure for the precise determination and recording of the location of structures in reference to the surrounding terrane, and also for the establishment of permanent reference points on the main structure which can be used in the future for determining any movement of the component parts of the structure.

More thought should be given, by those in charge of the construction of projects involving a change in the use of a terrane, to the possibility of the

[8] Prof., Civ. Eng., Carnegie Inst. of Technology, Pittsburgh, Pa.

[8a] Received by the Secretary April 29, 1940.

isostatic adjustments incident thereto and to making provisions for the determination of the amount of such changes.

In the paragraphs preceding "Triangulation" the author presents eleven precautions to eliminate constant and systematic errors and to minimize accidental errors. These items contain the key to success for any survey of this type. By carefully outlining the procedure followed in the work, Mr. Hough has brought to the attention of his readers the necessity for maintaining a constant alertness in order to eliminate all systematic errors and to prevent, as nearly as possible, the entrance of accidental errors into the work.

The details of procedure as outlined in paragraphs (*f*) and (*g*) of the "precautions" should be considered with care by all engaged in similar work. These paragraphs outline procedures whereby all errors incident to the instrument construction and operation can be either eliminated or brought to an extreme minimum. In paragraphs (*h*), (*i*), (*j*), and (*k*) personal errors have received careful treatment to make possible their elimination or reduction to a minimum.

Practically the only thing overlooked seems to be the possibility of using short-wave intercommunication sets, similar to those used by the U. S. Forest Service, for transmitting signals between the instrument man and the sight tender.

The description of the sequence used in the establishment of the main alinement points AW, AE, AE1, etc., as well as the placing of this line on the various monolith bars, should be treated at greater length. It is hoped that when the author makes his closing statements he will describe in detail the program followed in setting all points by use of the longest foresight available.

C. L. Garner,[9] M. Am. Soc. C. E. (by letter).[9a]—An excellent description of the accuracy that can be obtained with modern instruments used in measuring horizontal angles, and for alinement projects of the type described, has been given by Mr. Hough. There is no doubt but what the locations of points by alinements obtained with present-day optics by visual observation over short distances, combined with direct measurements, are much better than can be obtained through methods of triangulation which involve unusual refinement in many steps of the work, including reading of the circle graduations of the instrument. Stated in another way, the method of alinement described is a means of using the theodolite merely as a high-grade sighting device, thus eliminating some of the errors that always exist (even if they are small) in the graduation of theodolite circles and in the lack of perfection in setting up the instrument and the objects to be sighted upon at the various stations where the work is to be accomplished by triangulation. Undoubtedly, also, the method eliminates some of the errors of the personal equation.

On triangulation the difficulty of obtaining acceptable triangle closures involving short lines is due to the fact that, as the distances to the objects observed are shortened, the necessity of accuracy and refinement in the setting of targets and the avoiding of phase, eccentricity, or lack of symmetry becomes greater. All of these have a direct influence on the accuracy of the work as

[9] Commander, and Chf., Div. of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.

[9a] Received by the Secretary May 6, 1940.

evidenced by the triangle closures. Where they can be held in proportion to the accuracy sought, there is no question as to the capability of the instrument to furnish final results comparable to those which may be obtained through the alinement method. Where the lines are short or even a mile long, these problems become more and more difficult, and consequently the statement that "good triangle closures on extremely short lines are impossible" is true from the practical standpoint.

It is generally recognized that refinements necessary to obtain the highest accuracy where very short lines are involved outweigh the advantages of triangulation except where it is necessary to determine distances across streams or to inaccessible points. Where practicable, direct traverses are preferable.

For some years, the U. S. Coast and Geodetic Survey has employed first-order triangulation and traverses to detect small earth movements over areas of seismic activity. By repeating the observations from time to time as necessary, it is possible to determine progressive or other movements relating to the earthquake problem. Traverse measurements are used in locating the monuments that are closely spaced, and triangulation is employed in the location of monuments that are generally more than a mile apart. Thus it is possible to detect relative movements between consecutive monuments with an accuracy somewhat in proportion to the distance separating them.

In 1939–1940 the U. S. Coast and Geodetic Survey has cooperated with the Navy Department in alinement surveys in connection with its model basin project at Carderock, Md. This work was apparently quite similar to that described by Mr. Hough. On this particular project it was possible to obtain a higher degree of accuracy than ordinarily could be expected because the observations were made under ideal conditions in enclosed buildings of almost constant temperature, something rarely attainable in the field.

Incidentally, it is of interest to note that the theodolite described by Mr. Hough is an American product. A number of these instruments, the original of which was designed and built by the U. S. Coast and Geodetic Survey, have been in successive use since 1928. It has a 9-in. circle graduated to 5″ of arc and it is equipped with two micrometers 180° apart, which give readings of horizontal angle to single seconds. A distinctive feature of this instrument is its nonbinding center so that there is no binding or looseness in the movement of the horizontal circle due to temperature changes. For night observing, the illumination of the cross hairs and micrometer drums is provided through the center axis, the electric energy being supplied by the ordinary 6-volt dry-cell battery. The use of micrometer theodolites reading to one or two seconds of arc is not widespread in the United States. Such instruments must be manufactured on special order. However, during recent years, there has been a growing demand for more precise surveying instruments, and it is believed this demand will soon result in the wider use and adaptation of better surveying methods and improved equipment as necessary therefor.

A most important consideration in triangulation observations is that the instrument must be on a stable foundation, not subject to disturbance by the observer in walking around the instrument. In soft soil it becomes necessary to obviate this by installing a light platform, generally of 1-in. by 12-in. boards,

raised off the ground so that the observer's weight is supported at a distance considerably away from the instrument support itself. On precise observations it is also good practice not to change the focus of the telescope during observations. Some freedom of the parts is necessary in order that focusing can be accomplished, and the changing of focal length of the telescope during observations contributes to inaccuracy where short lines of different lengths are used.

Unquestionably the alinement survey principle has its advantages in locating points set for the purpose of studying movement in large concrete monoliths such as that contained in the Tygart Dam described by Mr. Hough. It is a most excellent means of determining small movements which may result from any cause whatever. To accomplish the same result by triangulation becomes more laborious as the scope of such a project is limited.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

NORRIS DAM CONSTRUCTION CABLEWAYS

Discussion

By Messrs. Walter F. Weber and Blair Birdsall, and G. E. Cate

Walter F. Weber,[5] Esq., and Blair Birdsall,[6] Assoc. M. Am. Soc. C. E. (by letter).[6a]—By the publication of this paper, a valuable chapter has been added to the meager literature on the subject of the design, construction, and operation of cableways. In particular, the writers wish to call attention[6b] to Equation (2*a*). In general terms this equation may be defined as an approximation of the path of load curve for a single load traversing an anchored span.

The two end points and the center point of this curve are usually fixed by the conditions of the problem (see Fig. 10(*a*)). To find any other point accurately requires the tedious solution by trial and error of a pair of catenaries which intersect at the load.

The conditions of Fig. 10(*a*) establish the over-all catenary length of the cable. Since the physical characteristics of the track cable are known or assumed, the unstressed or ground length of the cable may be found. This unstressed length remains constant for all positions of the load on the span, except for changes due to variation in temperature. To obtain any point on the path of load accurately, two intersecting catenaries must be found. These catenaries have known spans, but unknown rises. By trial and error, a horizontal component of cable tension which is common to both catenaries and a sag to the intersection point must be found, such that the over-all unstressed length is correct and the algebraic sum of the vertical components of tension at the intersection point is equal to the known traveling load.

The aforementioned accurate solution is very laborious and, if the formula published by the authors is a close approximation of the true curve, it is very valuable indeed. The writers have compared values obtained by the authors'

Note.—This paper by R. T. Colburn, M. Am. Soc. C. E., and L. A. Schmidt, Jr., Assoc. M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by J. S. Foster, Esq.; and April, 1940, by Gordon H. Bannerman, M. Am. Soc. C. E.

[5] Dist. Engr. for Chicago, John A. Roebling's Sons Co., Trenton, N. J.

[6] Asst. Engr., John A. Roebling's Sons Co., Trenton, N. J.

[6a] Received by the Secretary April 5, 1940.

[6b] Correction for *Transactions:* Number the formulas for convenience of reference as follows: December, 1939, *Proceedings*, page 1751 and top of page 1752, Equations (1*a*), (1*b*), (1*c*), (1*d*), and (1*e*); and page 1752, below Fig. 10, (2*a*), (2*b*), (2*c*), (2*d*), and (2*e*).

formula with those found by the accurate method, mentioned herein, for a 150-ton cableway.

The characteristics of the cableway are as follows: W = 400,000 lb, w = 134 lb per ft, s = 1,256 ft, h = 72 ft, d (with load at center) = 111.36 ft, cable area = 35.2 sq in., cable modulus = 17.0×10^6 lb per sq in. Using these basic data for both methods, the points given in Table 9(a) were found on the path of load curve.

TABLE 9.—ORDINATES d TO PATH OR LOAD CURVE, IN FEET

(a) LARGER VALUES OF x			(b) SMALLER VALUES OF x		
Values of x	Accurate*	Approximate†	Values of x	Equation (2a)	Writers'
0	0	0	0	0	0
55.40	27.42	27.95	1	1.15	0.92
120.00	45.64	46.37	10	7.99	7.60
184.60	58.59	60.38	20	13.56	13.32
306.47	79.62	80.51	30	18.21	18.10
392.44	91.13	91.41	40	22.32	22.31
628.00	111.36	111.36	55.4	27.95	28.04
Tan ϕ_1	0.945	1.438	* Intersecting catenaries.	† Equation (2a).	

In this table ϕ_1 is the angle of the path of load curve at the head tower. From Equation (2a), tan ϕ_1 is found by placing x = 0 in the first derivative of d with respect to x.

Due to the large discrepancy in tan ϕ_1, some may feel a little uncertain of the values of d found by the formula, for small values of x. Its accuracy in this region may be demonstrated as follows: The writers have developed another approximate method of handling the problem, which is now considered obsolete because it is more difficult to apply than the authors' formula, and does not yield more accurate results. It is mentioned herein simply as a means of demonstrating the accuracy of Equation (2a). To apply the writers' method, it is necessary to find the true value of tan ϕ_1. Using tan ϕ_1 and the sag with the load at midspan, a part of an ellipse having these characteristics is then fitted to the span. Thus, the ordinates d are certain to be close to the truth for small values of x. A numerical comparison between the two methods is given in Table 9(b).

These comparisons clearly indicate that Equation (2a) is a very close approximation of the true path-of-load curve for an anchored span, and the writers hope that this demonstration will overcome, in this case, the reluctance with which many engineers use empirical, approximate formulas.

The writers suggest that the equation may be slightly easier to apply in the following form:

$$d = \frac{x\,h}{s} + \frac{\frac{x}{s}\left(1 - \frac{x}{s}\right)(w\,s + 2\,W)^2}{2\,t\left[w + 4\,\frac{W}{s}\sqrt{\frac{x}{s}\left(1 - \frac{x}{s}\right)}\right]} \quad \ldots\ldots\ldots\ldots (3)$$

In case it is desired to obtain a more accurate value for $\tan \phi_1$ than that obtained by the use of the authors' formula, for some reason such as the power requirement, or the total included angle over a tower saddle, the writers suggest the following method:

Find the characteristics of the track cable hanging under its own weight only. A close approximation of these values can be obtained by using the method described by F. C. Carstarphen,[7] M. Am. Soc. C. E.

From these characteristics find the horizontal and vertical components of the free cable tension at the support. To the vertical component add the weight of the moving load, algebraically. The quotient obtained by dividing this sum by the free cable horizontal component is a close approximation for $\tan \phi_1$.

In closing, the writers wish to emphasize that, where true accuracy is required, there is no substitute for the catenary solution. This requirement would occur, for instance, in the case where there is a close clearance between the moving load and some fixed object. However, in most cases, the comparisons shown in Table 9 indicate that the authors' formula is entirely satisfactory.

G. E. CATE,[8] ESQ. (by letter).[8a]—A formula for the deflection of the cable at the point of load is presented in this paper, the load being at any point other than midspan. This formula is not new, having been presented in 1909 by William Hewitt,[9] who used the principle of an ellipse with major axis equal to the span and the minor axis equal to twice the deflection at the center of the span under loaded conditions with the load at the center.

A slightly different approach to the derivation of this formula is as follows: In Equation (2a) of the paper

$$d_1 = \frac{x\,h}{s} \qquad (4a)$$

and

$$d_2 = \frac{x\,(s - x)\,(w\,s + 2\,W)^2}{2\,t\,[w\,s^2 + 4\,W\,\sqrt{x\,(s - x)}]} \qquad (4b)$$

The part d_2 of Equation (2a) applies to level spans and is approximate, but is sufficiently accurate for inclined spans if $\frac{h}{s}$ is small. The nomenclature of the paper is used in general, except that y = the deflection at any point of the load for level spans.

When the load is at the center of the span (level supports), the moment for a uniform load is $\frac{w\,s^2}{8}$; and,

$$t_0 = \frac{w\,s^2}{8\,y} \qquad (5a)$$

[7] "Aerial Tramways," by F. C. Carstarphen, *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 875.

[8] Associate Civ. Engr., Construction Plant Division, TVA, Knoxville, Tenn.

[8a] Received by the Secretary April 8, 1940.

[9] "Attributes of Curves Described by Moving Loads on Suspended Cables," by William Hewitt, *Industrial Engineering*, November, 1909.

The moment for a concentrated load is $\frac{W\,s}{4}$; and,

$$t_1 = \frac{W\,s}{4\,y} \qquad \text{(5b)}$$

Consequently the total load is

$$t_0 + t_1 = t = \frac{w\,s^2}{8\,y} + \frac{W\,s}{4\,y} = \frac{s\,(w\,s + 2\,W)}{8\,y} \qquad \text{(6)}$$

For the general case of a load at any point x, the moment for a uniform load is $\frac{w\,(s\,x - x^2)}{2}$; and

$$t_0 = \frac{w\,(s\,x - x^2)}{2\,y} \qquad \text{(7a)}$$

The moment for a concentrated load is $\frac{W\,(s\,x - x^2)}{s}$; and

$$t_1 = \frac{W\,(s\,x - x^2)}{s\,y} \qquad \text{(7b)}$$

Similarly to Equations (5), the total load is

$$t_0 + t_1 = t_x = \frac{w\,(s\,x - x^2)}{2\,y} + \frac{W\,(s\,x - x^2)}{s\,y} = \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\,y} \qquad \text{(8)}$$

which is the general equation of the curve described by the path of the loaded point referred to rectangular coordinates with the origin at either point of support.

From Equation (8):

$$y = \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\,t_x} \qquad \text{(9)}$$

Let t_0 = the horizontal component of tension due to the cable alone, which is equal to $\frac{w\,s^2}{8\,y_0}$, y_0 being the deflection at the center of the span; and let t_1 = the horizontal component of tension due to the load alone. Then, Equation (9) may be written in the following form—(in which y_0 = the deflection due to the cable alone and y_1 = the deflection due to the load):

$$y_0 = \frac{w\,(s\,x - x^2)}{2\,(t_0 + t_1)} \qquad \text{(10a)}$$

and

$$y_1 = \frac{W\,(s\,x - x^2)}{S\,(t_0 + t_1)} \qquad \text{(10b)}$$

The total deflection equals $y_0 + y_1$; t_0 is a constant and the problem is to find t_1 in terms of known values; and t_1 varies from $\frac{W\,s}{4\,y}$ with a load at the center of the span to 0 if the load is at either support. Then if y' represents the ordinate of the intervening points of the curve,

$$t_1 = \frac{W\,(s\,x - x^2)}{y'\,s} \qquad (11)$$

Choosing an ellipse with a major axis equal to s and a minor axis equal to $2\,y_0$ as a gaging curve (this is not the curve of the path of the load), the equation of the ellipse is

$$\frac{x^2}{\left(\frac{s}{4}\right)^2} + \frac{y^2}{y_0{}^2} = 1 \qquad (12)$$

with the origin at the center of the span. Translating the origin to either support, Equation (12) becomes

$$y' = \frac{2\,y_0}{s}\sqrt{s\,x - x^2} \qquad (13)$$

and $\left(\text{since } t = \frac{s\,(w\,s + 2\,W)}{8\,y_0}\right)$

$$\frac{2\,y_0}{s} = \frac{w\,s + 2\,W}{4\,t} \qquad (14)$$

Finally,

$$y' = \frac{(w\,s + 2\,W)}{4\,t}\sqrt{s\,x - x^2} \qquad (15)$$

Then substituting the value of y' from Equation (15) into Equation (11), and simplifying:

$$t_1 = \frac{4\,t\,W\,(s\,x - x^2)}{s\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}} \qquad (16)$$

and, since (from Equation (6)) $t = \frac{w\,s\,(w\,s + 2\,W)}{8\,y}$,

$$t_0 = \frac{w\,s^2}{8\,y} = \frac{t\,w\,s}{w\,s + 2\,W} \qquad (17)$$

Equation (9) may be written in this form

$$y = \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\,(t_0 + t_1)} \qquad (18)$$

Substituting Equations (16) and (17) in Equation (18),

$$y = \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\left\{\dfrac{t\,w\,s}{w\,s + 2\,W} + \dfrac{4\,t\,W\,(s - x^2)}{s\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}}\right\}}$$

$$= \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\left\{\dfrac{t\,w\,s^2\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2} + [4\,t\,W\,(s\,x - x^2)]\,(w\,s + 2\,W)}{s\,(w\,s + 2\,W)\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}}\right\}}$$

$$= \frac{(w\,s + 2\,W)\,(s\,x - x^2)\,(w\,s + 2\,W)\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}}{2\,t\,\{w\,s^2\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2} + [4\,t\,W\,(s\,x - x^2)\,(w\,s + 2\,W)]\}}$$

$$= \frac{(w\,s + 2\,W)^2\,(s\,x - x^2)\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}}{2\,t\,\{w\,s^2\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2} + 4\,W\,(s\,x - x^2)\,(w\,s + 2\,W)\}}$$

$$= \frac{(w\,s + 2\,W)^2\,(s\,x - x^2)}{2\,t\,(w\,s^2 + 4\,W\,\sqrt{s\,x - x^2})} \quad \text{.................. (19)}$$

If y is replaced by d_2 and $(s\,x - x^2)$ by $x\,(s - x)$, and Equation (4a) is added, Equation (19) becomes Equation (2a), the formula presented by the authors.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

STANDARDS OF PROFESSIONAL RELATIONS AND CONDUCT

Discussion

By Messrs. C. Frank Allen, Arthur W. Consoer, George C. Ernst, Frank S. Bailey, S. A. McCosh, John H. Meursinge, Karl W. Lemcke, John Sanford Peck, E. D. Ayres, F. E. Turneaure, Fred Asa Barnes, and C. A. Mead

C. Frank Allen,[26] Hon. Am. Soc. C. E. (by letter).[26a]—Possibly the first item in any ethics provision is that the engineer should have the proper qualifications; that is, qualifications for success; and the writer was glad to find that Professor Mead had given the formula derived from Professor Mann's investigation of the elements of success.[4] Some years ago, while visiting in Zurich, Switzerland, the writer learned of an address that was made by a professor of a great school of engineering in Paris, France, in which five qualifications for success were presented. These qualifications had a little different slant from those of Professor Mann and were character, intellect, scientific foundation, knowledge of man, and knowledge of engineering. It is useless to discuss the question as to whether character stands at the head. It is there.

To how many, at first thought, does character mean just honesty? But character is much more than simple honesty. To the writer courage stands out; and the best synonym that he has been able to find for character is dependability, which is emphasized by Professor Mead. Taken together, character and dependability provide a picture far better than either, alone. To character the writer would assign courage, steadfastness, persistence, vigor, and adaptability. Even honesty of purpose lacks something if it is not combined with an acute sense of right and wrong. It is not necessary to enlarge on intellect.

Note.—This paper by Daniel W. Mead, Past-President and Hon. M. Am. Soc. C. E., was published in January, 1940, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Messrs. Louis E. Ayres, Ivan C. Crawford, Walter H. Wheeler, Charles R. Gow, J. T. L. McNew, and W. L. Waters; and April, 1940, by Messrs. Charles F. Scott, M. J. Evans, R. L. Sackett, Alonzo J. Hammond, A. B. McDaniel, C. B. Burdick, John M. Hayes, and G. W. Howard.

[26] Prof. Emeritus, Mass. Inst. of Tech., West Roxbury, Mass.

[26a] Received by the Secretary March 7, 1940.

[4] Address on "Engineering Education," by C. R. Mann, *Proceedings*, Am. Soc. C. E., February, 1916, p. 98.

The creation of a scientific foundation is clearly the function of the college of engineering. It is not necessary that an engineer should get his scientific foundation through the college; but he must get it in some way. It is somewhat uncertain how readily one can accede to the precedence followed between knowledge of men and knowledge of engineering. Regarding this subject someone has said, "Knowledge is power." That saying has not satisfied the writer as an engineer. It is the use of knowledge that is power. Coal is not power; but when it is used it generates power. Water in a still pond or reservoir is not power; but put into action it yields power. So it is the use of knowledge that is power; but when it comes to the use of knowledge, what can one do with it except in connection with men?

In the Society for the Promotion of Engineering Education some years ago there was a paper presented, "Why Not Teach About Men?"[27] and it produced a profound impression upon the writer's mind. Why not study men? Certain rules can be laid down, among them that one can better understand a man if he is observed when he is off his guard. The important point which the writer wishes to make is that if the young engineer will develop the habit of studying men, he will find that his ability in this direction will grow apace.

Useful knowledge should be based on scientific foundation. It should be sound, and not be fallacy. Then it should be exercised intelligently, and beyond that, it should be exercised righteously, with character.

In practice the engineer must depend very largely upon common sense and judgment; but from the writer's point of view common sense and judgment should be based upon the five qualifications stated. Common sense and judgment are not crude affairs but rest largely on experience and precedent—experience acquired and precedents accepted, both of which should have a scientific foundation and not be lacking in the other four qualifications for success.

The writer would like to see rules of conduct formulated for teachers of engineering in their relations to students. There are relations of engineers to contractors, and relations to others, but the relation of teachers to their students very well merits attention.

The young engineer who is placed in charge of a construction job should enter upon it with the presumption that the contractor he is dealing with is fair and honest. In the long run, with the majority of contractors, he will not be disappointed. If, in a few cases, he finds himself mistaken, here again knowledge of men may well come into play. Then as the work progresses, it is the function of the engineer to cooperate with the contractor by staking out work conveniently, by arranging the necessary sequence of operations, and in other ways by giving opportunity for the contractor to do his work successfully. This should operate to the advantage of the engineer's employer or client. Cooperation on one side invites return cooperation from the other side. The writer likes to see the contractors make money but never at the sacrifice of the quality of the work. He was raised as a strict constructionist of the specifications. In the matter of small favors from the contractor, he personally prefers that it should not be altogether one sided. It suited him for the contractor to smoke about as many of his cigars as he smoked of the contractor's.

[27] "Why Not Teach About Men, the Most Important and Difficult Tools an Engineer Uses?" by John F. Hayford, *Proceedings* of the Society for the Promotion of Engineering Education, Vol. XIV, 1906, p. 198.

As stated in the paper, there is a popular saying to the effect that opportunity knocks but once. The writer does not believe in that at all. It knocks often and one needs to hear the knock every time. Furthermore, there is something of value in the proposition that can be extended. The successful man is the one who takes advantage of his opportunities rather than the man of greater talent who lacks that quality. The writer feels confident that he can trace success due to this quality in two conspicuous cases: One of a man of not outstanding ability who became President of the United States; and another of a man of exceptional ability who became President and an Honorary Member of the Society. Along the same line, the writer advises young engineers not to tie up with the man that is constantly unlucky. Perhaps it is not luck at all. It has been said that a teacher should have a sense of humor and be an optimist. This is not restricted to teachers. In the line of optimism, it is good psychology both to build on, and to appeal to, the best in men.

There is a chance to improve the understanding of engineers in legal matters. The writer is profoundly impressed with the fact that a lack of knowledge of elementary principles of law on the part of the engineer may be a serious handicap in many cases; and he feels certain that an engineer should know enough of the rules of evidence so that he can collect evidence at the only time when opportunity offers and know what kind of evidence will stand up in court. The engineer should also know much about the laws governing contracts.

In closing, the writer wishes to express his appreciation of the suggestions, examples, and illustrations that are made, which engineers should take to heart. Engineers should become so saturated with the spirit of this paper that it will go a long way to building up a character which will enable them to do the right thing under the various circumstances that occur, and are not here enumerated.

ARTHUR W. CONSOER,[28] M. AM. SOC. C. E. (by letter).[28a]—Engineers in private practice will find Professor Mead's paper on professional conduct most interesting and informative. He has developed his subject so well that it is probably presumptuous for a younger engineer to take any exception to what he has written. However, it is the writer's opinion that, under present conditions, very few consulting and practicing engineers in the municipal field can be guided by his recommendation that no solicitation of professional engagements be undertaken (see code "IV. The Personal Relations of the Engineer," item, 19). Unfortunately, solicitation in general practice is rather widespread, and most engineers in general practice probably find it absolutely necessary, if they are to have practice sufficient in amount to enable them to maintain adequate staffs.

Many phases of such solicitation are discreditable in the extreme, and every effort should be made by engineers in general practice to discourage criticism of competitors, the use of shady influence, and other ignoble sale-agent tactics. Certainly bidding for engineering work should be discouraged, particularly in cities where it is evident that low price is to be a considerable factor in the selection of engineers. It seems to the writer only practical that for some years

[28] Pres. and Gen. Mgr., Consoer, Townsend & Quinlan, Chicago, Ill.

[28a] Received by the Secretary March 18, 1940.

to come solicitation of professional engagements by engineers in general practice must be condoned, but that until such time as that practice can be abandoned every effort should be made to remove from methods of solicitation all unsavory aspects. There is room in the Society for a division of engineers in general practice within which round-table discussion could probably be used to minimize objectionable features of solicitation; and gradually the situation could be improved so that ultimately all engineers in general practice could operate without resorting to active solicitation of new engagements. However, it will always be difficult for competent engineers to enter the field of general practice if they are to be estopped from active solicitation of clients, and perhaps some middle ground must be developed for the benefit of younger engineers who are desirous of entering private practice.

George C. Ernst,[29] Assoc. M. Am. Soc. C. E. (by letter).[29a]—It is inspiring to read this paper outlining the basic principles for a successful life. The encouragement of patience, diligence, and dependability, combined with independent and critical thought, should promote a steadying influence in these somewhat trying times. The faith that is still held in these qualities by the leaders in the profession will provide the backbone for the future development of the profession of engineering. It scarcely seems necessary to point out that every engineering student beyond the sophomore year should be required to study and analyze this paper.

Those connected with academic matters will find it difficult to read this contribution without becoming aware of the obligation and responsibility which rest upon the engineering faculties and schools in the proper training of students in engineering. Meticulousness in the details of pedagogy and attempts to teach excessive amounts of subject matter during the formative years of the 4-yr curriculum shrink in importance when the need is seen for properly preparing each student for the growth of the qualities described in the paper. Without the belief in such qualities implanted in the younger members, the profession cannot look forward with much hope and enthusiasm.

The engineering schools not only stand practically alone in their opportunity to supply the profession with men ingrained with the proper ideals, but should recognize it as their duty and privilege. The writer believes this type of training to be of unusual importance in the utilization of engineers in public service as well as in private practice and in industry.

The mere presentation of fundamental subject matter on the part of the teacher and its assimilation by the student may be accomplished by the routine methods now advocated and used by teachers of education in the handling of large groups. It should be obvious that such methods cannot adequately initiate the development of the proper ideals in the student to the extent known to be desirable in engineering. The writer is firmly convinced that there is a strong tendency on the part of many institutions, inadvertently, to permit or promote the growth of the student body at the expense of training in those very qualities which are esteemed by the profession. The necessity for having well-organized, adequately manned, and fully equipped classes and

[29] Asst. Prof., Civ. Eng., Univ. of Maryland, College Park, Md.

[29a] Received by the Secretary March 21, 1940.

laboratories should be evident if engineering teachers are to develop the requisite ideals (as well as training) for an engineering career.

It is evident that this need should not be construed to mean a highly specialized curriculum or a decrease in the dissemination of "scientific-technological" knowledge for the benefit of the so-called "engineering fraternity."[30] The profession of engineering, however, is entitled to a better initial selection of entering students than the present educational mortality of more than 50% indicates. The crowded conditions and the heavy mortality occurring in the first two years take a heavy toll of student morale, and the junior and senior students are adversely affected to a greater extent than many are willing to admit. The present graduates would be far better material if they were to start as a more compact group in the first year without the company of those leaving before the junior year.

FRANK S. BAILEY,[31] ASSOC. M. AM. SOC. C. E. (by letter).[31a]—In the "Synopsis" preceding the author's admirable paper, it is stated that "The paper has been written more particularly for the younger men of the profession and with the hope that it may supply them with some information, usually not available in college, as to what their action and conduct should be in their professional relations in practical life."

Almost all young men who have been taught elementary principles of good conduct, and who are under the necessity of earning their own living, and possibly of assisting others, enter their first employment with the determination to do their work well and to win advancement. Some succeed brilliantly, others only partly, and some to a very minor degree.

In looking back on their youth and middle age, many older men who have not reached the goals for which they aimed in youth may recall various periods in their lives when they might have used their working and spare time to better advantage. A careful reading and weighing of the precepts for success which the author has presented should benefit any young man who desires and intends to succeed. The writer also considers this paper a most useful one for older men who may be inclined to "rest on their laurels," instead of continuing to give their best efforts to whatever work they may be fortunate enough to obtain.

Some men are naturally born leaders and cannot find any comfort in a subordinate position, but the majority of men, after they have reached or passed middle age without demonstrating exceptional ability, become more or less reconciled to the idea that they do not possess the talents which enable men to rise to the higher positions.

Some boys have an unfortunate start in life. They may be given sound advice by some people and very poor advice by others at a time when they lack sufficient knowledge to distinguish between the two kinds. As they grow in experience they realize that the responsibility for any foolish decisions they make rests largely on their own shoulders, and if they are reasonable they learn to exercise great care in avoiding useless risks of all kinds. By the time they approach the end of their careers they probably acquire sound principles of

[30] "Report of Committee on Aims and Scope of Engineering Curricula," *The Journal of Engineering Education*, March, 1940, p. 555.

[31] With Metcalf & Eddy, Engrs., Boston, Mass.

[31a] Received by the Secretary March 26, 1940.

living. Of course, the earlier such principles are acquired the better it is, but "better late than never."

It is the writer's opinion that in the course of their lifetimes men are likely to reach the positions to which they are entitled, although there may be exceptions to this rule. It seems evident, nevertheless, that men who are naturally gifted as leaders are as subject to heavy blows from fate as those not so well endowed, and they may have to summon more courage and fortitude to survive some of their difficult experiences than those whose lot in life is more commonplace.

The author's paragraph on "Intelligence and Native Ability" recalled to the writer that, while reading the *Journal* of Ralph Waldo Emerson a few years ago, he was struck by the thought that Emerson, at the age of nineteen or twenty, possessed more practical acumen than more normal men attain at twice that age.

The writer feels that Professor Mead has rendered a noteworthy service in presenting this paper.

S. A. McCosh,[32] Assoc. M. Am. Soc. C. E. (by letter).[32a]—This paper places in the possession of the engineer something of inestimable value. The author, in his "Synopsis," states that the application of the principles set forth will result in the greatest personal satisfaction and success. The writer would magnify that statement to say that their practical application will insure, to a large degree, the accomplishment of the participant's desire or goal of life itself—that is, happiness and contentment, the universal desire of human beings, regardless of how they may express such desire or conceive the fulfilment of it. The worth of such application lies in the fact that it cannot be done on a purely selfish basis. Practise of principles, like the invention or development of something worth while, has an unlimited and unending possibility of human benefit.

There is only one exception to the truism that "nothing is constant except change," and that is the permanence of principles. An engineer's productive worth may end with his retirement or death; but his influence, of which he can never possibly know the full value, will continue long after he has departed. The often unconscious acts, mannerisms, etc., of the older, experienced engineer influence those who see and contact him, to an extent that he seldom realizes or comprehends. The younger engineer, to a lesser degree, is exerting the same general influence.

The principles of justice, fair play, honor, confidence, loyalty, and self-control, applied with common sense, will result in greater happiness for, and more efficient work by, engineers. To make principles practical requires more thought, study, and time than the most complicated technical problem. Witness the slow reduction in traffic fatalities in spite of the efforts of many organizations to make the United States safety-conscious.

John H. Meursinge,[33] Assoc. M. Am. Soc. C. E. (by letter).[33a]—Since time immemorial the inhabitants of this planet have been forced to keep up a great

[32] Asst. Prof., Civ. Eng., Colorado School of Mines, Golden, Colo.

[32a] Received by the Secretary April 8, 1940.

[33] Huntington Park, Calif.

[33a] Received by the Secretary March 29, 1940.

number of ever-changing rules, ordinances, laws, and codes in order to make an ever-changing society function properly. Inasmuch as the engineering profession is taking a place of increasing importance in present-day society, its need for a code of ethics is obvious. The elaborate paper by Professor Mead should prepare the way for the adoption of such a code. The standing of the profession, the success of the aims of its organizations, and the engineers' usefulness to society as a whole depend entirely on honest dealing with one's fellowmen. History, unfortunately, has proved that times of stress, such as the present, tend to lower (or often destroy altogether) professional morals. The publication of Professor Mead's paper, therefore, can be called timely.

The lowering of morals is a result of the lack of social security; and as, presently, the younger engineers are subject to more "layoffs" than the more mature, experienced engineers, Professor Mead fortunately wrote his paper particularly for the younger members of the profession. However, he fails to mention the fact that steady jobs will probably do much more for the ethics of these engineers than a well-written code. Although a great part of unethical dealings, undoubtedly, may be attributed to greed, another not so often mentioned part is caused by the fear of poverty and losing one's job. Hence, there is a possibility that a code of ethics published for a profession which has not reached the goal of social security yet may become as weak a structure as a building without a foundation.

The great majority of engineers are employees in the service of a public administration or of private companies. The type of work that many engineers are engaged in makes the permanence of employment rather doubtful. Excuses for cessation of employment can always be found in spite of any corrective regulations that may exist—especially since no powerful societies of engineers take an interest in social welfare. The engineers are like an army scattered over too large a territory. They are poorly entrenched and at the same time are on the firing line; there exists no general staff which directs the defense of the individual soldiers who are subject to attack. Therefore, an engineer who likes to do some clean fighting in regard to some ethical problem which has arisen in connection with his work may feel that the commotion brought forth by such an action may eventually cause the loss of his job. Indeed, only very strong characters under such conditions will value professional standards higher than the welfare of a happy family. Outside of the office or the project there has existed for the last few years a kind of a no man's land in which, during the greater part of the time, hundreds of unemployed engineers compete for the few available jobs in the same fashion as so many hungry dogs quarrel for a few chewed off bones.

Maximum practical results of a code of ethics can be expected only in a well-organized society in which the engineer with a family depending upon him does not have to face the privations connected with unemployment. Hence, the engineer, who is a practical man, in order to improve the morals of his profession should join the millions of Americans who today are so earnestly striving to obtain a social order which provides a living space for all. To make his influence felt the engineer must be able to talk intelligently and in an unbiased spirit about social security and economic problems of importance. His reading

and education should not be limited to technical subjects. Engineers should establish themselves as a progressive force respected by public opinion not only for their technical accomplishments, but also for their constructive interests in social and economic problems. Then the weaker brothers of the profession will be members of engineering societies which protect their jobs, their salaries, and their standing; and surely they will not be tempted so much to stray off the straight road of professional conduct.

Once the engineering societies have done their share in establishing social security for their membership, another problem related to ethics should be dealt with. Too many unethical dealings are, and have been, nothing but short cuts to fame and wealth. Success has generally been defined too poorly; and too often it has been expressed in the size of the engineers' salaries or in the amount of publicity some engineers happened to attract. Better norms to measure the dimensions of successful living will have to be found. Professor Mead presents a short essay on the subject written by the late Thomas Van Alstyne (28),[33b] which shows that the meaning of the term success has begun to broaden out beyond material gains. As the definition cannot be expressed in terms of the engineers' cherished mathematics, no two engineers probably will ever agree upon the correct expression. Nevertheless, more and better definitions, published from time to time in the engineering magazines, may improve professional conduct. Of all definitions published thus far, the writer likes the following best:[34]

> "It [success] will be judged by the immaterial and more consumable goods we have learned to enjoy, and by our biological fulfilment as lovers, mates, parents, and by our personal fulfilment as thinking, feeling men and women. Distinction and individuality will reside in the personality where it belongs, not in the size of the house we live in, in the expense of our trappings, or the amount of labor we can arbitrarily command. Handsome bodies, fine minds, plain living, high thinking, keen perceptions, sensitive emotional responses, and a group life keyed to make these things possible and to enhance them; these are some of the objectives of a normalized standard."

Summarizing, it should be stated that the engineers who believe in a code (which is something more than a list of rules fit for framing and hanging on the wall) should fight for:

1. A social order in which competent engineers will find continual employment;
2. The adoption of a code of ethics as outlined by Professor Mead; and
3. A profession which measures the success of life not by the tangible values taken from society, but by the moral values given to society.

[33b] For reference to numerals in parentheses, see "Bibliography of the Literature on Ethics and Human Engineering," in the Appendix of the paper.

[34] "Technics and Civilization," by Lewis Mumford, Harcourt Brace and Company Inc., New York, N. Y., 1934.

KARL W. LEMCKE,[35] ASSOC. M. AM. SOC. C. E. (by letter).[35a]—As stated by the author, "It is self-evident that all men desire success in their undertakings." Some, due to the possession of certain characteristics, are predestined to success, while others are not so fortunate. Just as the student in search of engineering knowledge can save himself much time by studying the knowledge accumulated by others and stored in textbooks, so too the engineer in search of help in building a career can learn from the experiences of those who have gone before.

In the "Synopsis" the author states that "The paper has been written more particularly for the younger men of the profession," and this the writer believes is rightly so. Throughout the early part of the paper, in particular, the young engineer will find considerable information of value, and the latter part will come to be more fully appreciated as he advances in years.

In this connection the subject of advice to the young engineer in the matter of a career, and certain suggestions as to the conduct of life, come so close to the author's subject matter that, were the title of the paper slightly different, the writer would like to comment on them. The writer wishes particularly to thank the author for his "A Code of Courtesy and Personal Conduct."

Under "Public Relations of the Engineer" and "The Personal Relations of the Engineer," "To do unto others as he would that others should do unto him" is a principle which, after testing for more than ten years in the selling of engineering materials for construction, the writer can heartily endorse. It pays far greater dividends than sharp practice, dishonesty, etc.

Referring to item 19, under "The Personal Relations of the Engineer," it would seem to the writer that, in this age of keen competition, under certain conditions it might not be unethical to solicit work actively, if done in a dignified manner. However, he admits that it would be more satisfactory to have the work come to one on the basis of his reputation and experience.

It seems to the writer that the author presents the case very succinctly when he states in item 12, under "The Engineer's Relation to Client or Employer," "A reputation for high ideals, good personality, honesty, dependability, intelligence, initiative, and professional ability is the foundation of professional preferment." This sentence might well be committed to memory by the young engineer.

Some time ago there appeared an article in one of the technical publications decrying those clauses in specifications which made the engineer arbiter or umpire between his client and the various contractors on a project, this on the ground that the engineer, being employed by his client, was not a disinterested party. Apparently engineers are living up to item 2 of the author's section VII (Relations of the Engineer with the Contractor), as there has been no change in this procedure, and all parties concerned seem satisfied to continue under the arrangement. This reflects great credit on the profession.

On occasion the writer has seen phrases in specifications in violation of item 9 of this section, but these could have been obviated by exercising sufficient care in writing the specifications and drawing up the plans.

---

[35] With Waddell & Hardesty, New York, N. Y.

[35a] Received by the Secretary April 1, 1940.

While selling engineering materials, the writer has seen many of the items of section IX (The Ethics of Contracting) grossly violated, particularly section *C*, items 2, 3, and 4. The author's code, if lived up to, would certainly place all contracting and subcontracting on a far higher moral and ethical plane.

The writer believes that every one could profit from the remarks in section X (A Personal Code of Conduct and Ethics). The code cited shows the young man to have been one of unusually high ideals. In the present state of things such young men are greatly needed in this world.

In conclusion, the writer feels that the engineering profession owes the author a great debt of gratitude for the writing of this paper. Just how much it will be worth to any individual will depend upon how much time he gives to the study of it and to the endeavor to practise its principles. One thing is certain, and that is that, if every member of the profession were to put its principles into use in his daily life, this world would be a far better place to live in, and the engineering profession would enjoy much more prestige than it does at the present time.

JOHN SANFORD PECK,[36] M. AM. SOC. C. E. (by letter).[36a]—The writer has always been opposed to fixed, rigid codes of any description, be they religious, moral, or professional. Excessive formalizing can lead only to stagnancy and sterility. In a too strict observance of the letter of the code, the spirit and heart are lost. Witness the fate of the Children of Israel in general and that of the Scribes and Pharisees in particular. Even Herbert Spencer, whom the author cites (10),[36b] held that "The sense of duty must diminish as moralization increases."

The writer places much more reliance in the modern philosopher's definition of "moral" as a guide to conduct. That action is "moral" which is based on a consideration of all its possible outcomes, and that outcome chosen which seems to be the best for all concerned. Every situation is unique. It never occurred in exactly that form before, and it never will occur exactly so again. Certain elements may be identical, but all elements are not identical, and hence no rigid code that gives all the answers can be constructed. Even the Ten Commandments are interpreted to fit practical conditions and are not applied literally. A prescribed code of ethics tends to dull the edge of the moral responsibility of the individual for the results of his actions, and transfers that responsibility to the code itself.

The author, while no doubt stating "good old-fashioned doctrine" (especially in that section devoted to advice to the young engineer), has failed to take cognizance of the fact that this is a changing world. What is regarded as the truth today may become the superstition of tomorrow. The social situation is changing by leaps and bounds. The college graduate of today demands something that will work, and he knows that most of the old maxims have failed badly "to deliver in the pinch."

---

[36] Asst. Prof., Civ. Eng., Coll. of the City of New York, New York, N. Y.

[36a] Received by the Secretary April 13, 1940.

[36b] Numerals in parentheses, thus: (10), refer to corresponding numbers in the Bibliography of the Appendix of the paper.

What profits it, if he have all the attributes set forth by the author and the only job he can get is changing nickels in a subway booth? He has discarded the old cult of the "go-getter," the "success" mania. The profession makes a grave mistake when it publishes a code that carries the implication that every engineering graduate is a potential leader of men. Napoleon said that every private in his army carried a field marshal's baton in his knapsack, but how many of them ever rose to wave it with authority over an army? History is silent on that point.

Instead, engineers should stress the satisfactions that may be achieved in life in the rôle of draftsman, transitman, or assistant engineer. The world has much more need of efficient and well-trained followers than it has of an embittered and disillusioned host of trained "leaders" who failed to achieve positions for which they were unqualified.

To turn to a few more specific comments, the writer is in total agreement with the author on the subject of engineering education (see heading "Introduction: Technical Training and Technical Ability"). The writer has labored for a broad foundation of fundamentals plus humanities in the curriculum, and has fought the trend toward too high specialization.

However, the writer must take issue with the author on the subject of the findings of psychology, especially with reference to individual differences. They cannot be thus lightly dismissed. Glands, the intelligence quotient (I.Q.), and inhibitions play a larger part in the life of the individual than the author will admit. Granted, a strong drive may overcome somewhat the handicaps of a low I.Q., but, on the other hand, a strong drive coupled with obvious inabilities may cause that individual untold misery and disappointment. At the writer's institution, a psychologist sits with the Committee on Course and Standing and has been very successful in diverting those students, who in spite of all personal incapacities want to be engineers, to other fields where they will be more useful.

Finally, the writer must take strong issue with the author when he cites the Golden Rule (26) and states that this is the basis of all sound ethical action. To presume to criticize this ancient law is taken by most laymen as evidence of atheism, radicalism and all other terrible isms; but there are sincere and earnest thinkers who doubt its reliability as an infallible guide. In a world where every personality was exactly the same, with the same likes and dislikes, the Golden Rule would be workable, but with the recognition of individual differences its infallibility disappears. Why force ice cream on a man who detests ice cream simply because you would like to be given ice cream yourself!

In conclusion, lest the writer be accused of being merely "a scoffer in the market place" and a low fellow who only drags down but does not build up, he would like to offer in place of the author's first part the moral of the Parable of the Ten Talents,[37] which is, in essence, the doctrine of individual differences. Each individual is responsible only for achievements commensurable with his abilities, but it is immoral to neglect even the smallest degree of ability. In

[37] Luke XIX: 12–25.

place of the author's complicated code of professional ethics, he would offer this quotation from a well-known modern philosopher: "Ethics, in my opinion, is not a matter of detached rules, but of a few simple principles. These are honesty in thought and action, courage, justice and the loving kindness preached by Christ, but often forgotten by Christians."

Finally, another philosopher, William Herd Kilpatrick, has said that it is better to raise a good question than to answer it. Why are codes necessary?

E. D. AYRES,[38] ESQ. (by letter).[38a]—Standards of professional relations and conduct are like international laws—they derive their force from the support of those to whom they apply, and they command respect by reason of their acceptance and particularly by reason of their observance by those whose standing among others commands attention. On the other hand, periods of stress force breakdowns in both professional codes and international law, and in each case the offending individual or nation seeks to rationalize his action.

The code presented is unusually complete and its content is comprised largely of almost axiomatic truths, just as much international law is based upon common sense and stands for justice to both big and little nations. Nevertheless, the tests of strength in codes and laws become most severe during the bitter periods of human existence. Although somewhat distasteful as an undertaking, it occurs to the writer that turning the discussion of this code in the direction of certain subversive comments, born of a depression era, may serve the useful purpose of inviting further discussion from queer angles, to the ultimate end of making the code proof against as many honest subversive views as actually may be encountered.

Standards of professional relations and conduct are always written with the young professional man, just emerging from school into his profession, particularly in mind. As it has been the writer's privilege to work quite intimately with many promising youngsters and to be in contact with them during some of their early struggles as junior engineers it seems particularly appropriate to try to discuss Professor Mead's paper as far as possible from their viewpoint.

As the period in the young engineer's life which the writer is trying to cover in particular might be called the "era of disillusionment," the approach to a code of ethics designed for his use will not be gentle. So true is this that only the successful engineer who is a living example of an ethical code can be sufficiently "strong medicine" to reach the aggressive young engineer passing through this era. For example, from the midst of a small circle of engineering students at the university not very long ago came this question, hurled like a hand grenade into an enemy trench: "Why don't they set up a practical course in engineering that will do us some good—a course in how to chisel the other fellow?" The questioner, during his Christmas vacation, had tried his hand for the first time in his father's business of selling motors and had lost his commission on an attractive sale through ignorance of the devious methods necessary (at least he was convinced that necessity was a major consideration)

[38] Chairman, Dept. of Eng. Economics, The University of Wisconsin, Madison, Wis.

[38a] Received by the Secretary April 26, 1940.

of getting around the Robinson-Patman Act. Words from his inexperienced fellow students or from a professor supposedly secure in his regular salary could not bring any soothing balm to this boy's wounds just then—only a father that played the game without winking at a Christian code of ethics or an eminent engineer in the game of selling, whose experience had proved that every turn in life's path must be navigated only by the light shed from the Golden Rule, could possibly make the crooked path for this boy for the time seem unattractive.

The foregoing incident reveals only too decidedly the need for the widespread endorsement of a living code and even more the dire need for successful engineers, through engineering societies and technical clubs, to get down among the young engineers and give from their very hearts something of their own devotion to the highest and best in professional practice in every devious professional path. It means also that, to make the proposed code a living code, the leaders in the fields of engineering selling and in the fields of engineering design and governmental practice, who have the status of regular employees of private corporations and governmental units, must rise to the occasion now and support the consulting engineer in the endorsement of this all-engineering code.

The need for a wide and adequate response from the successful engineering employee is brought out by the remark of another young engineer, employed in promotion and selling for one of the nation's largest trade associations, referring to this code and its author: "He never had the experiences of a struggling salesman. Any code is not going to reach us until it comes from someone whose experience parallels ours!" The justice of this criticism may be questioned, but the disillusioned young man attempts to be realistic, and to reach him codes must get down to where he is, not so much in content, but through the endorsement and living example of leaders in the very field in which the young man works.

Turning to a few of the specific clauses presented in the code, and again approaching the task from the view of the young engineer in the "era of disillusionment," items IV–6, IV–17, and IV–19 offer some opportunity for perversion in meaning. As written, the youth feels strongly that there is an element in these parts of the code put there only to protect the "vested interests" of the older established engineer. As the younger man seeks a job in the trying times of yesterday and even today, he is confronted on every hand with the demand for experience. He finally wonders how he is to obtain this experience if he is denied the chance to begin somewhere. Such a youth reading item IV–17 reads into the code at this point simply another bar put up to block his progress. With this view as a background he proceeds to read into item IV–6 that he is to be barred ethically from competing with the older established engineer because certain legitimate recourses to aid from others, which are open to all, are ruled out. Proceeding in this same frame of mind to item IV–19, he finds there that all that is left to him in the form of positive action is forbidden him by an ethical code—that is, he must not even solicit professional work.

No one knowing the author of this paper will question the fact that the "slant" just offered was far from his mind when those clauses were written. Nevertheless, some amplification of these same clauses with deference to the evil "slant" just brought out perhaps should be attempted in the endeavor to perfect the code for general use. Every practicing engineer can produce much evidence to support the work of, and the need for, the sense of these same clauses. A few sentences expanding their meaning to avoid any construction of their content as unfavorable to the young engineer just getting his bearings seem desirable to protect the axiomatic good that lies in them.

In closing, the writer would like to make the comment that only widespread publication and discussion of a code can perfect it and fasten it into the minds and hearts of the profession. It seems to be a duty for some, who are in contact with special groups, to bring to the surface all manner of criticisms of this code which they find rising to them spontaneously from sincere interpretations of its contents. It stands as a monumental work. If the code successfully can stand trial in the fires of widespread discussion, the profession can have a code carrying the authority of an universal acceptance which can be expressed explicitly.

F. E. Turneaure,[39] Hon. M. Am. Soc. C. E. (by letter).[39a]—Professor Mead has done a valuable piece of work in formulating in such detail the standards of professional relations and conduct which should be followed by members of the profession. The particular attention he has given to the student and the young engineer will be of especial value, not only to this particular group but also to those in the colleges who are concerned with their education.

To one who has been in contact with college students for many years and who has followed the later careers of many of them, it seems certain that such a detailed formulation of principles will be of great help to the beginner in reaching correct decisions in the analysis of his personal problems. Not infrequently the young engineer is confronted with situations involving questions of professional conduct that are somewhat difficult to analyze rightly and which, if wrongly answered, may lead him into further and more serious difficulties.

The increased attention being given this general subject in engineering schools is encouraging, but to be very effective the instruction should be given by men of wide experience as otherwise it is likely to be looked upon as too academic and idealistic to be of much practical value.

Opinions differ as to the quantitative significance of the percentage tables of characteristics quoted by Professor Mead but he has made it clear that the primary purpose of the quotations is to emphasize the vital importance of character and not to minimize the necessity for adequate technical knowledge. These two phases of the engineer's equipment are difficult to compare. They are both essential.

[39] Emeritus Dean, Coll. of Mechanics and Eng., Univ. of Wisconsin, Madison, Wis.

[39a] Received by the Secretary May 8, 1940.

FRED ASA BARNES,[40] M. AM. SOC. C. E. (by letter).[40a]—It would be useless to attempt to add materially to this monumental paper; but the writer would like to subtract something from it. He cannot refrain from expressing his regret that Professor Mead has revived and republished the results of the so-called "investigation" of engineering education.[4] Of course, the author takes the pains to point out ("Introduction: Ability to Apply Principles to Practice") that "those factors common to educational requirements; * * * are absolutely essential to the success of the engineer," as do other writers who have made use of these data. For example, Howard Lee Davis in his book, "The Young Man in Business," states, "Of course, the employer cannot well place a man in a position for which he lacks the necessary education and experience." In the light of this comment, what do the results shown in Table 1 mean?

In the opinion of the writer they are the statistical outcome of the effort, probably unintentional, to mislead the 30,000 members of the Founder Societies into using their emotions in the rating of essential characteristics. That is, every one has a feeling, or emotion, that character and integrity are of great importance, and so they are; but what would one do if asked to "rate" his vital organs in "order of importance"? Is it not significant that less than one quarter of the engineers addressed replied at all and that replies from only a little over one sixth of them were sufficiently definite to be tabulated? It seems to the writer that, even if there were any logical and reasonable basis for rating essentials, fixing their relative importance to the nearest 0.1% (and apparently for all time) on the basis of so small a proportion of replies is fanciful in the extreme.

Furthermore, even if the results were valid, would they have much significance with reference to engineering education? Most thoughtful people realize the necessity of the characteristics mentioned in all lines of endeavor, but the writer would like to point out that students come to the colleges and universities with their characters and other personal traits rather thoroughly crystallized—that is, their development is largely the responsibility of the home and the preliminary schools. The college should and usually does continue the training along ethical lines, both by example and by formal courses, and Professor Mead's paper will long be a source of valuable material for such purposes.

C. A. MEAD,[41] M. AM. SOC. C. E. (by letter).[41a]—The young engineer is likely to set aside the Code of Ethics adopted by the Society as applying to those in established private practice, even though the use of the word "client" is noted to include "employer." Professor Mead, in his timely paper, has expanded these rules so that all ranks and positions in the profession are covered in their application. They are not hastily conceived; and they carry conviction because the principles have long been proved and tried and have been found dependable and enduring, as every older man knows.

---

[40] Prof., Railroad Eng., School of Civ. Eng., Cornell Univ., Ithaca, N. Y.

[40a] Received by the Secretary May 13, 1940.

[4] Address on "Engineering Education," by C. R. Mann, *Proceedings*, Am. Soc. C. E., February, 1916, p. 98.

[41] Upper Montclair, N. J.

[41a] Received by the Secretary May 13, 1940.

A former president of Hamilton College, in Clinton, N. Y., once defined character as "the habit of an attitude." Professor Mead rightly sets out character as the chief cornerstone in the career of any man and emphasizes the principles which are essentials in its formation. The use of the time a man makes of his "off" hours is a surer index to what he really is than those which are spent within his required hours of work. "Good measure, pressed down and running over" shows the manner of the man and will find recognition when the time for advancement comes.

The weight given to the characteristics necessary for the greatest success cannot be evaluated accurately, of course; but it is clear that all are agreed that character, judgment, and industry are three times as valuable as the knowledge of fundamental studies and technique, however valuable they are. These are matters on which the mind of the young engineer should be focused, and the earlier he assimilates them the sooner he will enjoy the freedom of success. They are the basic principles which a wise man cannot afford to ignore.

This paper states positively what can be done by any man and does not leave the student with a skeleton of negative limitations; it gives him most valuable aid in a most comprehensive way. It is worthy of a place in the curriculum of every college, where it should be reviewed frequently so that it may become the mentor of every student.

As each man is the director of his own destiny, he should make the most out of his life. Precept and example are his only teachers at the outset of his career; but if he is careful in the selection of them he will find it easier in his later years, when ripe judgment is required, to give the honest and full return his clients expect.

Professor Mead's paper is replete with sound ideas of good conduct which, if followed, will lead to a success which is enduring and which will bring the maximum of satisfaction to him who makes it his rule of life. It is a most valuable contribution to ethical relations of professional engineers.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

CHICAGO RIVER CONTROL WORKS

Discussion

BY MESSRS. A. E. NIEDERHOFF, R. P. V. MARQUARDSEN, ISAAC DEYOUNG, JOHN W. WOERMANN, EDWARD SOUCEK, W. C. WEEKS, AND HENRY R. KING

A. E. NIEDERHOFF,[3] ASSOC. M. AM. SOC. C. E. (by letter).[3a]—Several phases of the Chicago River Control Works not previously well understood are explained in this paper. The author has given an excellent and reasonably complete account of the history, design, construction cost, and operation of what has been termed "the lowest lock lift in the world."

On December 26, 1939, the writer made it a point to visit the lock at the mouth of the Chicago River to satisfy a growing curiosity as to the construction and framing of the sector gates and the arrangement of operating machinery. Upon arrival, the lock tender volunteered the information that the head on the lock from the lake side was 0.3 ft. However, upon opening the sector gates no water could be observed flowing from the lake into the lock chamber. If there was a head of 0.3 ft at that time, it completely escaped the attention of the writer.

The structures shown in Fig. 1 deserve some explanation as to how they perform the task of separating, "definitely and effectively, the Chicago River from Lake Michigan." The Government breakwater on the east side of the basin and the North Pier on the north side of the basin are not, and were never intended to be, impervious, watertight structures. Fig. 15 shows in cross section the type of breakwater used. The original construction occurred in 1870–1876 and consisted of dumped rock surmounted by a rock-filled timber crib. The top of the breakwater is a more carefully built structure with a concrete cap that, at last reports, was in good condition. In general, the percentage of voids in the rock mass and the permeability of the structure are great. Obviously, as these two sides of the basin now exist, they cannot possibly hold back a head of water for any sustained period. The question is raised as to how these pervious structures are to be converted into a watertight

NOTE.—This paper by H. P. Ramey, M. Am. Soc. C. E., was published in January, 1940, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[3] Civ. Engr., U. S. Army Engrs., U. S. Engr. Office, Portland, Ore.

[3a] Received by the Secretary March 25, 1940.

barrier. Certainly the project cannot be considered complete until some further remedial work has been done and the total cost increased accordingly.

The author mentions that several suggestions for protecting the lock walls from impact and rubbing by boats were ruled out because of the fear of small

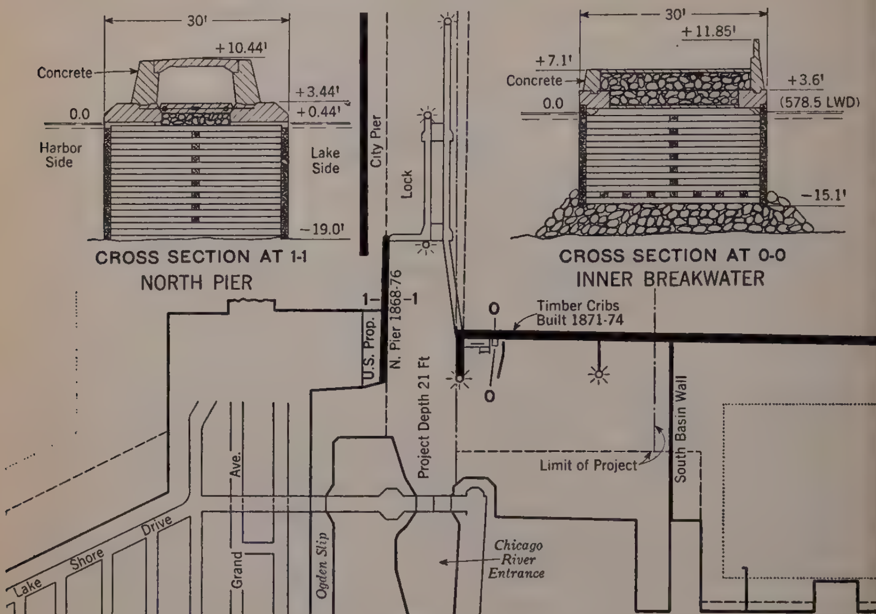


Fig. 15

boats tilting when they became caught under horizontal wales or because the usable lock width would be decreased by the width of the protecting element. These arguments are difficult to comprehend because the same problem of protecting lock walls has occurred repeatedly in the 20 years 1920–1940 and has been successfully solved in the structures on the Ohio, Mississippi, Allegheny, Monongahela, and other rivers. The U. S. Army Engineers have found it good practice and a necessary part of the cost of lock construction to armor all corners, ladder recesses, and even longitudinal rubbing strips on the guide and lock walls. Armor has been incorporated in lock walls of modern structures built in the past two decades. Rolled steel sections embedded in concrete are used, and they present a hard, durable surface against which boats can rub without ruining the concrete. The nominal lock-chamber width is frequently increased 1 in. between neat concrete lines to compensate for the fractional part of an inch taken up by the armor.

The spalled concrete that has been witnessed on the Chicago River lock, although not of structural importance but certainly unsightly, may be caused primarily by impact from boats. However, the appearance of the broken walls is such that there is doubt in the writer's mind that boat impact alone was responsible. Something more elementary during the construction phase may have been a contributing factor. Poor materials and workmanship

coupled with lax supervision during construction have resulted in other concrete structures deteriorating within a year in much the same manner that the lock walls have been broken up. The writer is well aware of the practice of gaging the strength and soundness of concrete by crushing "inspector-made" cylinders of representative concrete batches. This is an applicable criterion of partial suitability only when the same care that the inspector uses is duplicated in placing concrete in the forms. Even this guide is not an adequate substitute for good workmanship under strict supervision to obtain concrete capable of withstanding the ravages of temperature range and exposure encountered in Chicago Harbor. Alternate freezing and thawing, ice conditions, and the critical inspection of an interested public at the exposed location of the lock would seem to make it highly desirable to place in the structure the most durable concrete that engineers and technicians can obtain.

The description of the lock gates has cleared up the obvious question of how debris was kept from piling up on the roller track at the bottom of the gate. It is inferred that the combination of wire brooms and raised curbing in which the track is embedded has worked satisfactorily. The operating machinery with its vertical shaft motors and novel method of speed reduction should be described in greater detail. Perhaps an assembly view of the motors and machinery would indicate the desirable compactness of the unit.

Derivation of time required for filling, and of the rate of inflow, for various heads on the lock has been handled particularly well. The reasoning is substantiated by actual tests, and this in itself makes the paper valuable to those engaged in the design of navigation locks. Engineering literature is complete with theoretical analyses for filling various types of conventional locks, and the hydraulic laboratories have reams and reams of data on novel lock-filling and lock-emptying devices. When it comes to actual tests on the prototype, the investigator is able to find but few references. Even these references are almost entirely confined to tests on locks using the conventional through-wall culvert and wall port systems. These tests on filling a lock through sector gates are the first of their kind known to the writer, and the author's diligence and industry in reporting them are to be commended.

R. P. V. MARQUARDSEN,[4] ESQ. (by letter).[4a]—Mr. Ramey is to be commended for making available to the engineering profession the information contained in this paper. The description is general in character, but the author has managed to go into the various phases of the project in considerable detail, although perhaps not always to the extent that the specialist might desire in connection with the particular line in which he is interested.

Lock Walls and Guide Walls.—Little can be added to the description of the lock walls and the guide walls. The 1-in. mastic expansion joints provided in the reinforced concrete cap of both the lock walls and the guide walls were placed directly above an interlock between two adjacent sheet piles in the face arcs of the cells. To each of these sheet piles was welded, for reinforcement, a 10-in., 45-lb, wide-flange beam, 58 ft in length.

[4] Structural Designer, The Sanitary District of Chicago, Chicago, Ill.

[4a] Received by the Secretary March 26, 1940.

The main features of interest in these walls are:

(1) The arrangement of the diaphragms of the cells, which, as stated by the author, enabled the contractor to start construction at several points. This was a very desirable feature, inasmuch as the existing wave action made it unwise to drive many cells before commencing the work of filling them.

(2) The simple method of construction—the walls were constructed without cofferdams and without employing divers, the type of wall permitting all of the work to be done from above the water level.

(3) The exposed part of the steel sheet piles is permanently below water level, the top being encased in concrete down to low-water level.

(4) The lock walls are so designed that the entire lock chamber may be pumped dry should occasion require.

North Basin Wall.—The cells of the north basin wall were formed by steel sheet piles. All of the piles are 58 ft long and were driven to a bottom elevation of − 55.0, except those for the two end cells and for the diaphragms, which are 48 ft long and were driven to a bottom elevation of − 45.0 Chicago City Datum (C.C.D.). The average penetration of these piles is about 30 ft and 25 ft, respectively. The radius of the arcs of the piles forming the two faces of a cell in the part of the basin wall located north of the basin-wall gate block, or control-gate section, is 34 ft, except for the end cells, where a radius of 18 ft $3\frac{3}{4}$ in. was used. Two of the diaphragms are straight and two are curved, the radius of the curved diaphragms being 60 ft. The radius of the face arcs of the cell between the control-gate section and the west gate-foundation structure is 45 ft. At the intersection of each diaphragm with the face arcs, special fabricated Y-piles were used.

The north basin wall was designed as a gravity dam in a manner similar to the lock walls, the assumed conditions being as follows: (1) Base of wall, at El. − 26.0 C.C.D.; (2) width of wall at base, 34 ft; (3) water pressure to El. + 6.0 on one side and to El. − 2.0 on the other side; (4) no wave action; (5) weight of rock fill at 90 lb per cu ft in the dry and 65 lb per cu ft submerged; (6) angle of repose of rock fill, 45° (1 to 1 slope), corresponding to a ratio of horizontal to vertical intensity of pressure of 0.172; and (7) no surcharge.

South Basin Wall.—As stated by the author, the Z-piles used in the south basin wall were designed as vertical simple beams with a cantilever at the top to carry the horizontal load caused by the submerged rock fill and a head of 7.83 ft of water. The points of support were assumed at El. − 0.50 (the elevation of the horizontal wales and the tie rods) and at El. − 30.11 (the calculated first point of contraflexure below the lake bottom).

The wales were designed as a simple beam to carry the horizontal reaction of the Z-piles at El. − 0.50, the points of supports being the tie rods.

The tie rods were designed to take the reaction of the wales on the low-water side of the basin wall, the support of the tie rods being the wales bolted to the Z-piles on the high-water side. The Z-piles on the high-water side in turn bear against the rock fill of the wall, thus completing the combining of functions of the several parts of the basin wall that enables the wall to act as a gravity dam.

The south basin wall, whose north face is a continuation of the north face of the U. S. Naval Reserve, has greatly increased the docking facilities at that point, the wall forming the south limit of a sheltered basin.

The wall, in conjunction with the United States breakwater, also serves as a vehicular and pedestrian traffic connection link from the U. S. Naval Reserve to the U. S. Coast Guard station and to the south half of the lock.

Lock Gates.—The author mentions briefly the rubber sealing strips provided at the bottom and the two vertical sides of each gate-leaf face. These rubber strips serve to seal the gates when the lock is closed. The vertical edge of each gate-leaf face at the intersection with the recess side closes against a vertical structural-steel support embedded in the gate block. The other vertical edge (at the intersection of the gate face with the lock side) of a leaf closes against the corresponding edge of the companion leaf of the same gate. The bottom edge closes against the cutoff curb on the floor slab of the gate-foundation structure.

The rubber sealing strips at the bottom edge and at the recess-side vertical edge are of the so-called lamb-chop design, being shaped somewhat like a lamb chop with a $\frac{5}{8}$-in. hole in the enlarged portion. The over-all width is $4\frac{1}{8}$ in.; the general thickness, $\frac{3}{4}$ in.; and the diameter of the enlarged portion, $1\frac{3}{4}$ in. The section is reinforced with 4-ply, 18-oz ducking, which envelops an interior body of soft, pliable rubber, the outer rubber encasing the duck reinforcement being tire-tread stock.

The rubber sealing strips at the vertical edge closing against the corresponding edge of the companion leaf of the same gate are of a special design, the general shape being that of an arc having a radius of 1 ft $6\frac{1}{2}$ in. The over-all width of the strips is 15 in. The thickness at the center (for a width of 9 in.) is 2 in., and at the two sides, 1 in. The section is composed of two parts cemented together to form a single unit. The part placed against the structural steel is made of 15-in. belt backing reinforced with 4-ply, 28-oz ducking. The other part is reinforced with 4-ply, 18-oz ducking and covers a 1-in. layer of sponge rubber in the center width, the outer rubber encasing the duck reinforcement being tire-tread stock.

The rubber sealing strips are fastened to the gate leaves by means of $\frac{5}{8}$-in. bolts, spaced about 6 in. on centers, and 2-in. by $\frac{3}{8}$-in. continuous bars, placed on the outer face of the sealing strips. The rubber sealing strips of the special design are fastened with two rows of bolts and two continuous bars, one at each side.

Concrete Gate Blocks.—Each gate-foundation structure supporting the two leaves of a lock gate, as the author states, is of rigid reinforced concrete construction and consists of two gate blocks (one for each of the two gate leaves) rigidly connected to, and resting on, a reinforced concrete floor slab, the distance between the inner faces of the gate blocks being 80 ft, the clear width of the lock chamber. The floor slab, including its untreated foundation piles and surrounding steel sheet piles, is described quite fully by the author.

Each gate block consists of a certain number of reinforced concrete walls and slabs, so arranged as to provide the required features. The general dimensions and elevations are given by the author. Some of the features common

to each gate block are: (1) A gate-leaf recess; (2) an operating-strut chamber; (3) an operator's house; (4) electrical manholes; (5) two tile gages for indicating water levels; (6) connections for two temporary curved cofferdams of single rows of steel sheet piles, extending from gate block to gate block; and (7) two connections for a temporary straight cofferdam enclosing the gate-leaf recess.

The gate-leaf recess, as stated by the author, is shaped like a 60° sector with a radius of 52 ft 6 in. This radius is for the wall of the recess, the radius for a cantilevered curb at the top of the gate block being 50 ft 9 in. The recess is used for housing the gate leaf when the lock is open. The details of the gates are such that the face of the timber wales on the lock side of the leaf is 8 in. behind the face of the gate block when the leaf is housed.

The top of the floor of the operating-strut chamber slopes toward the gate-leaf recess, the elevation varying from + 2.0 C.C.D. at the entrance to + 2.75 C.C.D. at the rear. The top of the roof is at El. + 7.0, except over the operating pinion and the structural-steel carriage where the elevation is + 10.0. The floor is provided with a horizontal concrete runway in which the track plates for the operating-strut casters are embedded at El. + 3.0. The operators' houses are described quite fully by the author.

Two tile gages for indicating water levels are set in the lock-side faces of each gate block, one on each side of the gate-leaf recess. The top of the top marking of the tile gages is at El. + 6.0 and the bottom of the bottom marking is at El. − 8.0, thus allowing for a variation in level reading of 14 ft. Each foot is divided into tenths, the numbers and the graduations being navy blue and the field white. The gages are made of standard bright-glazed ceramic tiles.

The connections for the cofferdams consist of steel piles cast in the gate blocks. They are located in small recesses in the gate-block face and extend vertically from the top of the gate block to the top of the floor slab, one interlock protruding beyond the concrete surface of the recess, for connection with the end sheet piles of the cofferdams.

The two temporary curved cofferdams consisting of single rows of steel sheet piles and extending from gate block to gate block are for use when it becomes necessary to make repairs to both leaves of a gate. One cofferdam is located on each side of the gate and thus they provide, in conjunction with the gate blocks, a complete gate enclosure from which the water may be removed and in which the gate may be repaired in the dry. Each row of steel sheet piles forming a cofferdam is assembled to a radius of 48 ft $1\frac{3}{4}$ in.

The temporary straight cofferdam enclosing the gate leaf is for use when it becomes necessary to make repairs to a single gate leaf. The cofferdam consists of a single row of Z sheet piles supported at the top against a double structural-steel girder and at the bottom against the concrete curb on top of the floor slab, already mentioned by the author. The cofferdam, when in place, covers the entire lock-side opening of the gate-leaf recess and thus provides, in conjunction with the walls of the gate-leaf recess, a complete enclosure for one gate leaf, from which the water may be removed and in which the leaf may be repaired in the dry.

A special feature for the northeast gate block is a boat stall, 6 ft wide by 10 ft high by 29 ft 3 in. long, the bottom of the stall being at El. − 4.0. The

boat entrance of the stall is provided with mooring bars, and with ladder rungs for ingress and egress of the boat operator through a manhole in the floor of the northeast operator's house.

The principal features of the gate-foundation structures, worthy of mention, are:

(1) Each gate-foundation structure, being designed as a rigid structure, insures the uninterrupted functioning of the gate leaves, even if the structure, through settlement at one end or side, should tip slightly as a whole toward that end or side;

(2) The sheet-pile connections in the gate blocks connect the temporary steel sheet-pile cofferdams and provide enclosures for each individual gate leaf or for each gate. The proper combination of these cofferdams can also be made to form a single enclosure for both gates and the entire lock chamber;

(3) The details of each gate-leaf recess are such that the gate leaf cannot overrun while closing, the vertical structural-steel support embedded in the concrete work for closing contact with the rubber sealing strips at that edge of the gate face being adjusted to act as an automatic stop when the gate leaf is in a fully closed position; and

(4) The provision in the northeast gate block for a boat stall in which is kept a boat for use in cases of emergency, the boat stall being within the lock chamber when the lock is closed.

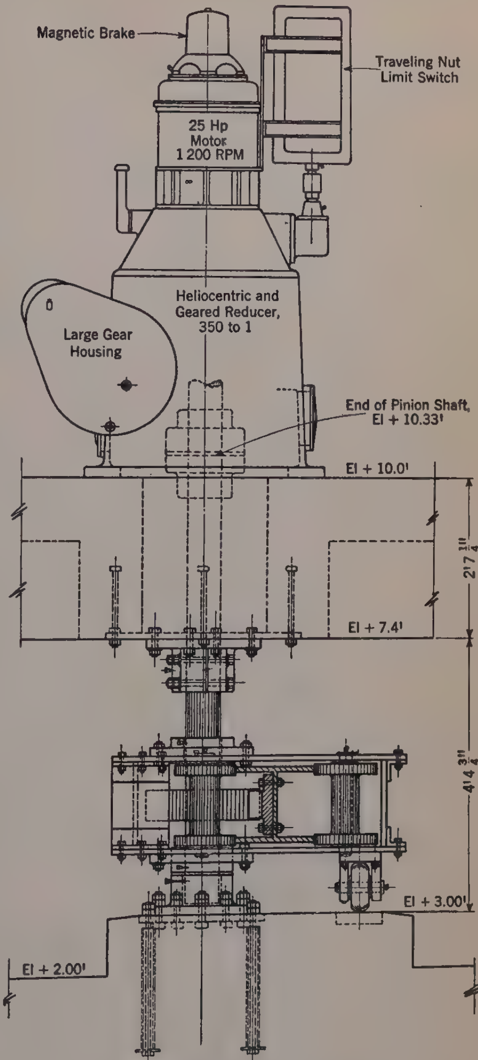


FIG. 16.—ELEVATION VIEW OF POWER UNIT

Operating Machinery.—The machinery for operating the lock gates is of a very ingenious design and, in the writer's opinion, deserves a more detailed description. The operating machinery for each gate leaf consists essentially of a power unit and an operating unit. The assembly of the operating machinery is shown in Figs. 16 and 17.

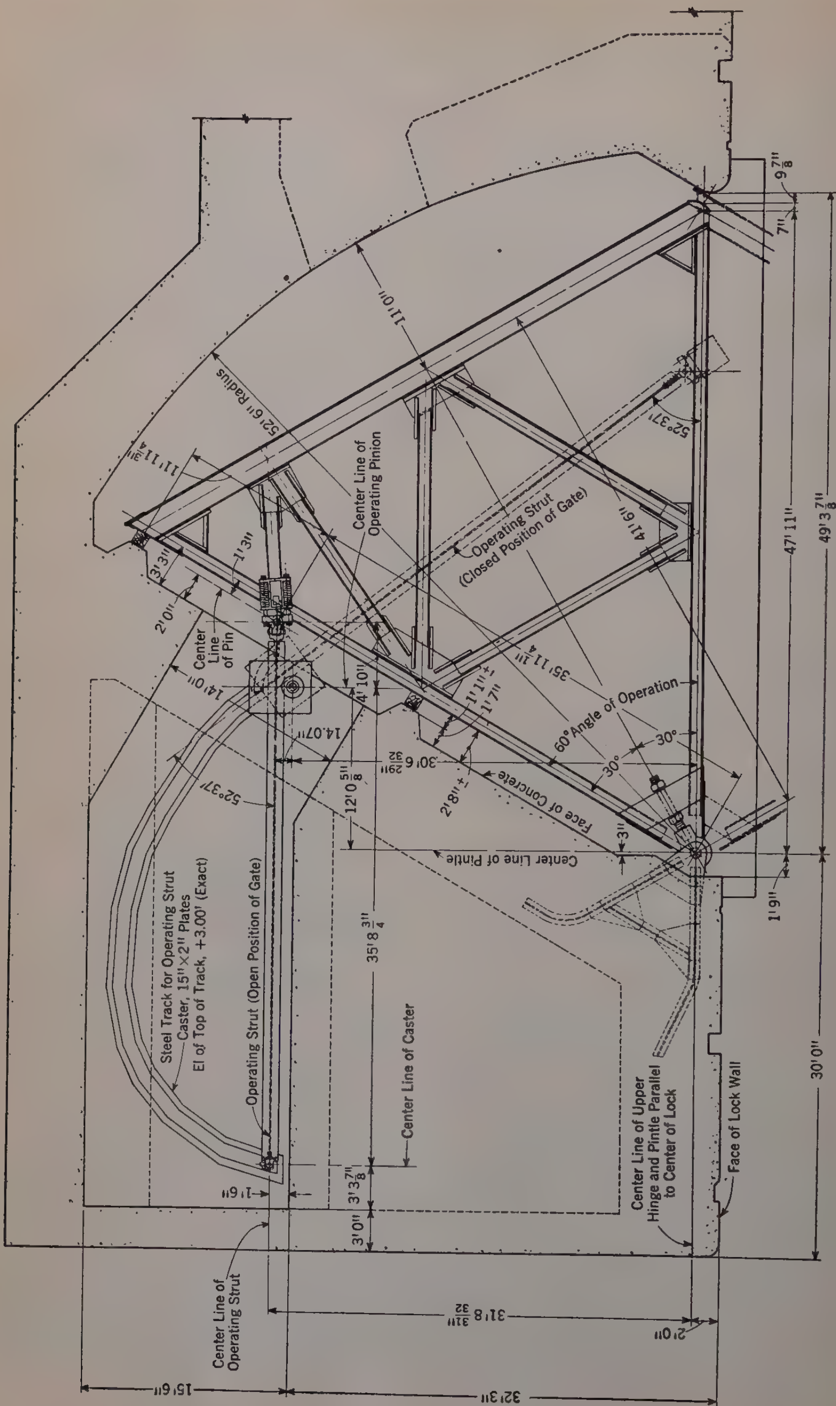


FIG. 17.—LOCATION OF OPERATING STRUT

The power unit (see Fig. 16) has two motors: A large motor for high-speed operation, which is a 25-hp, 1,160 rpm motor; and a small motor for low-speed operation, which is a $1\frac{1}{2}$-hp, 1,145 rpm motor.

A 350 to 1 reduction (for high-speed operation) between the output shaft of the large motor and the output shaft of the power unit is obtained in two steps as follows: First reduction (5.846 to 1) through a set of planetary helical gears meshing with a pinion on the output shaft of the large motor; and second reduction (60 to 1) through the internal mechanism of a large heliocentric reducer whose input shaft is connected to the output shaft of the planetary helical gears. The output shaft of the heliocentric reducer is also the output shaft of the power unit.

A 30,000 to 1 reduction (for low-speed operation) between the output shaft of the small motor and the output shaft of the power unit is obtained in four steps as follows: First reduction (4.529 to 1) through a small helical gear meshing with a pinion on the output shaft of the small motor; second reduction (40 to 1) through the mechanism of a small heliocentric reducer whose input shaft is connected to the output shaft of the small helical gear; third reduction (4.03 to 1) through a large helical gear meshing with a pinion on the output shaft of the small heliocentric reducer; and fourth reduction (41 to 1) through a worm wheel, which is bolted on the external face of the internal rack of the large heliocentric reducer, meshing with a self-locking worm on the output shaft of the large helical gear. The worm wheel, when in operation, connects with the output shaft of the large heliocentric reducer, which (as stated previously) is also the output shaft of the power unit.

Because of the self-locking effect of the worm drive, the design of the power unit is such that both the large and the small motors may be operated simultaneously. This feature makes it possible to cut in the large motor while the small motor is still operating, which means that the strut load to be picked up by the large motor is smaller than it would be if the small motor had to be cut out before the large motor could be cut in.

If only the small motor is to be operated, it becomes necessary to make the internal mechanism of the large heliocentric reducer inactive. This is done by applying a magnetic brake mounted on top of, and connected to, the large-motor shaft.

The output shaft of the large heliocentric reducer (and of the power unit), to which the vertical shaft of the operating unit is attached by means of a flexible coupling, thus revolves at a higher or lower speed depending on whether the large or the small motor is being operated, the higher speed of the shaft (and of the operating pinion) being 3.32 rpm and the lower speed 0.0382 rpm, for the aforementioned reductions.

A limit switch of the traveling-nut type, providing six sets of contacts and mounted in a cast-iron frame with a gasketed cover, is bolted to the frame of the power unit. Its mechanism is connected to the output shaft of the large heliocentric reducer through a set of 5.5 to 1 helical gears, resulting in a minimum amount of backlash. The vertical shaft of the operating unit is supported in a vertical bearing, bolted to the concrete gate block.

The operating pinion keyed to the shaft is of forged steel and meshes with a cast-steel rack bolted to a horizontal structural-steel operating strut, which is provided with a caster at one end and is hinged to a spring buffer connected to the gate leaf at the other end. The caster runs on track plates set in the concrete gate block, at El. + 3.0 (see Fig. 17).

The spring buffer consists of a cast-steel frame attached to the gate leaf, four steel springs ($18\frac{1}{2}$ in. free length) pre-stressed to remain inactive under a tensile or compressive stress of as much as 30,000 lb in the operating strut and held in place by four shafts (passing through the springs), and a two-way hinge casting for connection with the operating strut. The buffer is provided with a limit switch with spring return to trip out the starting contactor of the operating motors in case of excessive stresses in the machinery. The switch, which is mounted in a watertight cast-iron case, is operated by means of a lever moved by the change in position of the spring buffer. One unit of the switch opens when the total compressive stress in the operating strut reaches 45,000 lb and closes when the stress is reduced to normal. The other unit of the switch opens when the total tensile stress in the operating strut reaches 45,000 lb and closes when the stress is reduced to normal.

A structural-steel carriage, which is provided with a caster roller running on a segmental plate set in the concrete gate block at El. + 3.0, turns on the vertical pinion shaft and holds the rack in mesh with the pinion, the rack being kept in proper position by four vertical guide rollers forming part of the carriage. The rollers, two on each side of the operating strut, turn against the top and bottom flanges of the strut to which the rack is bolted.

The sequence of operation of the operating machinery in closing and opening the gates is described quite fully by the author and needs no further elaboration. Attention may be called to the compactness of the power unit, which occupies approximately an area of 4 ft by 6 ft and stands only about 8 ft above El. + 10.0, and to the fact that the operating strut and rack and the operating pinion and structural-steel carriage are all above El. + 4.0, the normal high-water level of Lake Michigan.

Miscellaneous Structures.—Among the many miscellaneous structures and structural devices provided in connection with control works and not referred to by the author the following may be mentioned: (1) Reinforced concrete stairs in connection with the crest walls, (2) ladders, (3) mooring hooks, (4) mooring rings, (5) check posts, (6) capstans, (7) pipe handrailing, (8) manholes and handholes, and (9) life preservers.

Reinforced concrete stairs with pipe handrailing, for getting from one side of the crest walls to the other, are provided at several points, there being two such stairs on the southwest guide wall, three on the south lock wall, and three on the southeast guide wall.

Ladders for descending from, or ascending to, the top of the lock are provided at several points, there being three on the lock-side face of each of the lock walls, five on the southwest guide wall, six on the southeast guide wall, two on the northeast guide wall, and two on each of the four gate blocks. The ladders consist of $1\frac{1}{2}$-in. galvanized-steel ladder rungs set in 1-ft 6-in. by 1-ft 6-in.

vertical grooves in the face of the concrete work, from the top of the lock to the bottom of the concrete work, which, in the case of the gate blocks, means to the top of the floor slabs. Ladder rungs of appropriate design are also set in the face of the concrete walls of the gate blocks, at various points.

Mooring hooks for the temporary tying-up of motor boats and other smaller craft while in the lock are provided at several points, there being eight in the lock-side face of the lock walls and two in each of the gate blocks. The mooring hooks are located at El. + 2.0 in an 8-in. deep recess, 1 ft 5 in. wide by 2 ft 3 in. high. The mooring hooks are galvanized and shaped approximately like an S. They are forged from steel rods $2\frac{1}{2}$ in. in diameter.

One mooring ring is anchored at the east end of each of the two east guide walls and one at the west end of the southwest guide wall. The rings have an inside diameter of 6 in. and the eyehook anchor is 2 ft long, both parts being galvanized and forged from 2-in. steel rods.

Check posts for mooring large vessels while in the lock, or while waiting to pass through the lock, are cast in and anchored to the concrete work at El. + 7.0, at about 65-ft intervals on each side of the lock and guide walls (except on the south side of the south lock wall, where there is only one check post at each end) and at 86-ft intervals on the lock side of the gate blocks, there being two check posts per gate block. The check posts are of cast steel and are 1 ft high above the top of the base, which is 1 ft 3 in. square and 3 in. thick. The top of the base is set flush with the top of the concrete, each check post being anchored by means of four $1\frac{1}{2}$-in. bolts, 1 ft 9 in. long. The body of the check posts is 10 in. in diameter at the top and $5\frac{1}{2}$ in. at the bottom.

A hand-power capstan for use in opening or closing the gate leaves manually in cases of emergency is provided at the top of each gate block. Each capstan is anchored securely to the gate block and is provided with four wooden capstan bars for turning.

Pipe handrailing is provided at critical points over the entire lock: (1) On the lake side of the northeast and northwest gate blocks and on the west side of the northwest gate block, (2) around the gate-leaf recess of each gate block, (3) along the north side of the north lock wall and of the northeast guide wall, and (4) around the inside of the walks on each gate leaf. The handrailing consists of two $1\frac{1}{2}$-in. pipe rails and 2-in. pipe posts, connected together and in place with malleable fittings and flanges, the flanges being cinch-bolted to the top of the concrete work. The handrailing is 3 ft 6 in. high, the distance between the pipe rails being 1 ft 6 in. and the pipe posts being spaced about 5 ft on centers. All of the handrailing parts are galvanized.

Electrical manholes and handholes are constructed in the gate-foundation structures and in the reinforced concrete frame of the north lock wall and northeast guide wall, at required points. The manholes and handholes are provided with frames and covers, and two pulling irons are installed in each manhole.

Life preservers in steel cases with glass fronts are provided at 12 points in the lock: Two on each side of the south guide wall, two on each of the lock walls, and one on each of the four gate blocks.

Construction Features.—As stated by the author, no particular difficulties were encountered in the construction of the lock. The steel sheet piles for the

several cells of the lock and guide walls were driven in about 24 ft of water into the lake bottom, which is soft gray clay for the top 5 or 8 ft and hard gray clay of increasing stiffness below the soft clay.

Double-acting hammers, on floating rigs, were used for driving. The cells were driven to heavy timber templates. At starting points, the templates were supported, just above water level, by temporary framework consisting of cross-braced wood piles driven into the lake bottom. For succeeding cells, the templates floated on the water level and were held in place by being lashed to the cell previously driven. All of the steel sheet piles of a cell were assembled around the template before driving commenced, and they were all driven completely before work was started on a succeeding cell.

Although the construction of the walls was performed in water partly sheltered from storms, it was found inadvisable to drive too many cells before filling them with stone or gravel. Consequently, the work of driving the steel sheet piles for the cells and the delivery of the stone and gravel for filling the cells had to be coordinated in such a manner as to have a sufficient number of cells ready for filling when the stone or gravel arrived and yet not have them remain unfilled for too long a period.

The stone was delivered by large self-unloader boats, in lots ranging from 2,850 to 7,700 tons. The gravel was delivered in similar but smaller boats, in lots ranging from 1,900 to 4,100 tons. The stone and gravel were unloaded from the boats directly into the cells without rehandling and without any labor being performed in the cells.

Both steel and wood forms were used in the construction of the reinforced concrete frames encasing the top of the steel sheet piles of the lock and guide walls. The outside bottom and side forms were suspended from overhanging structural-steel frames resting on jacks placed on top of steel columns supported in the stone or gravel fill of the cells. Four steel columns were used in each cell, one in each corner. They were left in place and encased in the concrete frame. No bottom forms were used for the concrete work inside the cells. The side forms rested on the stone or gravel fill.

All concrete for the lock and guide walls and for the two gate-foundation structures was mixed at, and distributed from, a "pump-crete" plant located on top of the cofferdam for the west gate-foundation structure, at the southwest corner. The cement, sand, and gravel were delivered on, and used from, barges tied to pile clumps alongside the mixing plant, there being no material storage on the cofferdam except very small quantities for use in cases of emergency.

The upper hinge of each gate leaf was set and concreted in its gate block before the lower hinge was placed. The position of the pintle of the lower hinge was established by plumbing from the upper hinge. The lower hinges were set and concreted in recesses left in the floor slabs of the gate-foundation structures.

Two divers were employed in placing the pre-cast reinforced concrete floor slabs for the lock chamber. To minimize the required slab movement at the lock-chamber bottom, the location of the slab joints were marked on the walls (above the water level), and the boom of the crane was set to correspond before the slabs were lowered into the water. After the crane had lowered the slabs

to almost the lock-chamber bottom, the divers had little more to do than to square up the slabs just as they were being released.

ISAAC DEYOUNG,[5] M. AM. SOC. C. E. (by letter).[5a]—Works of the magnitude and importance of the Chicago River Control Works should be so built that there will be no likelihood that operations will be interrupted. It is unfortunate that ample funds were not available to permit the construction of the Chicago River Control Works in accordance with designs ordinarily used for such structures. There is no inexpensive method of constructing works of this nature and magnitude for permanence.

The established plane of reference of Lake Michigan is 578.5 ft above mean tide at New York City. The elevation of Chicago City Datum (C.C.D.) is 579.94 ft above sea level. The floor of the lock is at El. 24.94 C.C.D. and is therefore 23.5 ft below the plane of reference of Lake Michigan. The sheet piles of the lock wall cells were driven to − 55 C.C.D., or about 30 ft below the floor of the lock. The lock walls were designed to sustain a head of 29 ft of water. Although the lock will be rarely unwatered, should this be done there would be, theoretically, an uplift pressure of about 1 ton per sq ft under the floor of the lock. It is not believed that the unbalance of pressure will cause any concern, as the piling has been driven to the stiff clay stratum. However, since the material is plastic, it is believed that it would have been better if the floor of the lock had been made of reinforced concrete with a transverse sectional form of an inverted arch and anchored to the clay bottom by timber piling. It is certain that, in time, a considerable volume of debris will be carried into the lock chamber by the action of vessels' propellers. If the floor of the lock were made in the form of an inverted arch, the collection of the debris would be largely confined to the low parts of the floor and could be removed more easily by a derrick boat with clamshell bucket. Deep recesses near the gates are of a great advantage for the collection and confinement of debris.

It is an established fact that much material and debris are moved by the action of the propellers of vessels. Shoals are formed to a considerable extent at the approaches, as well as in the lock, and will require removal to maintain project depth.

It is noted that in less than one season of use there has been considerable spalling of the concrete faces of the lock. Since a concrete top is attached to the steel piling of the cellular walls of the lock, it is believed that the use of pressure concrete, shot by a nozzle, will give a mixture of much greater strength than form-placed concrete. The compressive strength of such a mixture is from 6,000 to 10,000 lb per sq in. and is double that of ordinary form-placed concrete; less mass will be needed and no forms will be required.

To protect the present lock-wall faces from abrasion and further spalling, oak fenders may be attached to them, a three-strake oak fender being bolted to two ship channels fastened to the masonry with fox bolts. The oak fenders are placed in the troughs of the channels and the center oak strake is placed between

[5] U. S. Senior Engr. and Gen. Supt., St. Marys Falls Canal, U. S. Engr. Office, Sault Ste. Marie, Mich.

[5a] Received by the Secretary April 5, 1940.

the top and bottom channels. The maneuvering of vessels cannot be controlled to any extent.

Broken rock was preferred for the filling of the cells because of greater shear resistance. Gravel filling would be more compact and free from voids. It is believed that if timber piles were driven within the cells in conjunction with the gravel filling it would provide the desired shear. The added cost of the piling would, in part, be offset by the lower unit cost of gravel. Settlement of the rock filling will continue for a long time.

An excellent pavement may be made on the top and back of the piers and lock walls by using a mixture of tar and gravel, topped with road asphalt and covered by a thin layer of pea-sized gravel, the pavement being built about 3 in. deep. The tar and gravel are mixed in a small concrete mixer, using a mixture of 3 gal of tar and 7 cu ft of gravel. This mixture is spread over a leveled surface and compacted by tamping or by a hand roller. After rolling, the surface is covered with heated road asphalt, over which a thin layer of pea-sized gravel is spread.

In the matter of underwater lubrication of bearings, where low temperatures exist, the selection of a proper lubricant is important. It is found that a grease with a lead soap base has given excellent service. Chemically prepared graphite greases or oils have not given satisfactory service; the liquid vehicle is lost, leaving the graphite to solidify in the grooved bearings, thus preventing the lubricant from functioning. Because of the low temperatures, it requires considerable pressure to force the lubricant to the bearings.

JOHN W. WOERMANN,[6] M. AM. SOC. C. E. (by letter).[6a]—An excellent and detailed account of the design and construction of a new type of navigation lock has been presented by Mr. Ramey. It is particularly adapted to construction in deep water where a head is likely to develop at either end of the lock. This paper is particularly valuable on account of the widespread interest in this unusual lock. Letters of inquiry concerning various features of the lock have been received by the U. S. Engineer Office at Chicago from many other U. S. Engineer Offices in the United States.[6b]

Credit for suggesting the use of steel cells filled with rock for lock walls and guide walls, thus eliminating the necessity for an expensive cofferdam in 25 ft of water, is due Brig.-Gen. Max C. Tyler, M. Am. Soc. C. E., then assistant to the chief of engineers, U. S. Army. In connection with the design of this lock, General Tyler also suggested a hydraulic study, which was made under the direction of the writer, to consider the economics and feasibility of maintaining an artificial elevation of water surface in the Chicago River and connecting waterways of − 2.0, − 4.0, and − 6.0, Chicago City Datum (C.C.D.). This study showed that the − 2.0 project was the cheapest to construct and maintain. It was also the safest, considering the old wooden pile foundations along the river, because the water in the Chicago River had been at this elevation repeatedly, due to natural causes, for as long as six months at one

[6] Principal Engr., U. S. Engineer Office (*retired*), Chicago, Ill.

[6a] Received by the Secretary April 24, 1940.

[6b] Copies of the plans and specifications are no longer available for distribution.

time, without damaging the buildings that rested upon these wooden piles. The maintenance of an artificial water surface at − 2.0 C.C.D. would furnish additional bridge clearances for the nagivation of river barges and towboats which will be particularly valuable if a system of fixed bridges is adopted at some time in the future. The maintenance of the water surface at − 2.0 also furnishes very valuable potential storage capacity for the temporary storage of heavy runoff which is likely to pile up in the Chicago River several times a year while the hydraulic gradient is being increased between Chicago and Lockport, where the storm water would discharge into the Des Plaines River.

Referring to this artificial water surface of − 2.0, the author (see heading "General Conditions") states that "the differential in water levels may average 2 ft," which is true. However, at times of high lake level, this difference may amount to 5.0 ft, and on rare occasions to 6.0 or 7.0 ft. The author's following statement, "At present this differential is about 6 in., or less," means that at the time the paper was written the difference in level at the lock was 6 in., or less. Usually it is much less, because no effort will be made to establish a water surface of − 2.0 below the lock until the old Government breakwater and the U. S. North Pier are made watertight.

Stone was used in the lower part of the lock wall cells to reduce the internal pressure in the cells.[6c] This is important if it becomes necessary to pump out the lock. The filling consists of rock up to elevation − 2.0 C.C.D., as stated by the author (see heading "Walls: Lock Walls"). The gravel fill extends from − 2.0 to the top of the cell, which is at elevation + 7.0, a total distance of 9.0 ft.

The failure of one of the steel cells in the cofferdam during construction is ascribed, by the author, to the longer radius used in the arcs of the cofferdam cells and the omission of weep holes in the piling. These were no doubt contributory causes, but the character of the filling was undoubtedly a third cause. This consisted of clean, smooth, globular pebbles, about the size of marbles, which had an exceedingly flat angle of repose.

The author states (heading "Walls: Lock Walls"): "The advantage of the open concrete grill on top of the rock-filled cells, acting as a horizontal truss, over the same grill stiffened by a concrete slab is not apparent to many structural engineers. The difference in action, if any, should be very slight." The principal advantage, as seen by the writer, is the saving of concrete. The areas left open during concrete construction and subsequently filled with gravel and topped with limestone screenings constitute a very considerable portion of the total horizontal surface, as will be realized from an inspection of the plan view in Fig. 3. Furthermore, if there should be any settling of the rock or gravel filling, additional material can be added through these openings.

Lock Gates.—With the adoption of the steel cell design for the lock walls, search was started for a satisfactory solution for the lock gates. As stated by the author, "The requirement that these gates be capable of withstanding a head of water in either direction and also remain closed against surges from the lake eliminated ordinary miter gates from consideration." It was necessary

[6c] Correction for *Transactions:* In "Cross Section of Lock Wall" in Fig. 4, change "Gravel Fill" to "Rock Fill"; and, on page 77, line 19, change "11 ft" to "9 ft."

also that the lock be filled and emptied from the ends of the lock instead of through culverts in the lock walls in the usual way. Consideration was given to the use of two pairs of mitering gates at each end of the lock, one pair pointing toward the lake and one pair pointing toward the land, but this was considered too complicated and expensive. Consideration was given also to the use of a single reversible pair at each end of the lock (without a miter sill) so that the gates could be pointed in either direction, but this was considered too uncertain and dangerous. Other forms were given consideration, including sector gates of the type used in the Blue Island lock on the Calumet-Sag Channel. This type admits water only between the gates, along the center line of the lock, and creates considerable disturbance to a tow in the lock.

A more desirable type of sector gate was found in the new lock at Södertälje on the Södertälje Canal, about 42 km (26 miles) southwest of Stockholm, Sweden. This lock is described in detail in a report prepared by Sal. Vinberg and Lars Lawski, Swedish Government engineers, for the Thirteenth International Congress of Navigation held at London in 1923.[7] In describing the lock and some of the conditions that had to be met, Messrs. Vinberg and Lawski state that, although the lock gates are required to resist only a small head, this head is exerted sometimes from one side and sometimes from the reverse side. Another condition to be provided for was the escape through the lock of a large quantity of water during the spring floods. The locks accommodate ships drawing from 21 to 23 ft of water, carrying 6,000 to 8,000 tons when fully loaded. The water depth on the sills is 24 ft 7 in., the height of the gate 32 ft 1.73 in., the width of opening 65.62 ft, and the clear length of the lock, 442.91 ft.

Consideration was given to the use of roller gates, double mitering gates, reversible mitering gates, locked mitering gates, bascule gates, and sector gates. Referring to the sector gate, Messrs. Vinberg and Lawski say that it is a requirement for the best and most continuous filling of the lock that the greater part of the inflowing water should pass around the gates. At Södertälje two thirds of the water is made to pass through the gate recesses. The water that flows through a gate recess is forced against the wall of the recess and runs along it as if it were flowing through a side channel. The three streams coalesce close below the gates, and thus both their kinetic energy and their velocity are eliminated, and the water flows smoothly and with but small velocity into the lock chamber.

After comparing the advantages and disadvantages of each of the six types of lock gates enumerated above, the authors conclude that, taking into account the lack of security in operation of locked gates and reversible mitering gates, and the high cost of construction and operation of double mitering gates, it is obvious that the final decision will be in favor of either sector gates or roller gates. Of these two, the sector gates are the cheaper; the operating procedure is simpler, as there are no sluice gates to operate; operating the gates is less complicated; the time required for filling and emptying the lock is shorter, giving increased capacity to the canal; the security in working is greater than

[7] "Dispositions or arrangements to be adopted for locks, inclined planes, and other means of overcoming differences of level with a view to facilitating the operations," by Sal. Vinberg and Lars Lawski, *Report No. 13* relating to Inland Navigation, XIIIth International Cong. of Navigation, London, 1923.

with the other types of gates, as sector gates can be operated in running water; and, on account of the strong and stable attachment to the masonry, ice and wind will have less effect upon them. For these reasons, it was decided to have sector gates for the lock at Södertälje. A plan sketch of the gates is shown in Fig. 18.

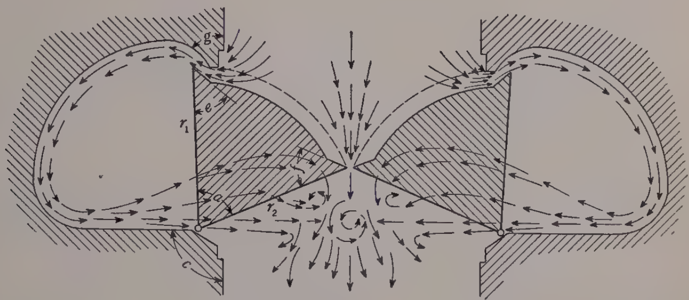


FIG. 18

As this type of gate was new, models were made to determine the best value for the center angle at the hinge of the gate (angle *a*, Fig. 18); the gate opening speed; the time required to take a boat through the lock with various lifts; the distribution of the inflowing water in regard to force and direction; stresses in the cables with which the boats were moored (parallel to the longitudinal direction of the lock); and other features. It was found: That, taking into consideration the cost, the center angle (*a*) should be 70°; that the downstream wall of the gate recesses should be at right angles to the face of the lock wall (angle *c*, Fig. 18); that the angles *e*, *f*, and *g*, Fig. 18, should be alike and equal to 20°; that the gate radii, r_1 and r_2, should be approximately equal; and that about one third of the inflowing water should pass between the gates and about two thirds through the gate recesses. Other characteristics were determined which are not of general application.

It was originally intended to place a driving gear on the wall, which would mesh into a rack along the outside plate of the gate, but on account of the possibility of unequal settlement in the masonry, and of vibrations in the gate, wire rope transmission was substituted. This construction also decreases the strain on the machinery on account of the elasticity of the wires and the spring connections between the ends of the wire rope and the gate. The springs become tense when the gate comes into its final position, and in this way help to overcome the inertia in opening the gate. There are two different motors working on equalizing gears, thus allowing the drum to be driven at different speeds.

The results obtained in these model studies of the sector gates at the Södertälje lock have been summarized briefly because they constitute the basis of the design adopted for the sector gates of the Chicago River lock.

The author's summary of the important features of the Chicago River lock gates; his description of the concrete gate blocks, operating machinery, electrical features, and sluiceways; his cost data, hydraulics of filling and emptying the lock; and his statement of benefits from the control works—are all extremely complete, interesting, and valuable.

EDWARD SOUCEK,[8] JUN. AM. SOC. C. E. (by letter).[8a]—This discussion is confined to the hydraulic calculations in that part of the paper headed "Operation of Lock." The writer has recently had to study a similar problem and was particularly interested in the formulas and coefficients used in the paper.

The author's formula for discharge is a form of the "Weisbach" equation without velocity adjustments. This equation has been used for calculating the obstructive effect of bridge piers but is now commonly considered to be theoretically incorrect for that purpose.[9,10,11,12] It has been shown,[13] however, that if no drop-down occurs upstream from, or within, the contraction, it is proper to use the Weisbach equation. Accordingly, the author's choice is believed to be correct.

By adopting the Francis formula for the "weir" term rather than the customary $\frac{2\,b}{3}\sqrt{2\,g}\,h^{1.5}$, the author has in effect applied the coefficient 0.62 to the weir term as usually written. When he applies an over-all coefficient of 0.95, the final formula is equivalent to the Weisbach equation with a coefficient of 0.95 for the orifice term and a coefficient of 0.59 for the weir term. It would seem preferable to examine the formula in this manner because the element of crest contraction—probably the essence of the value 3.33 in the Francis formula—is entirely lacking in the present case.

There is some evidence that a lower coefficient should be used for the weir term than for the orifice term in the Weisbach equation. An extensive series of experiments to determine the discharge between Chanoine wickets[14] has been performed by the U. S. Engineer Department in the University of Iowa Hydraulics Laboratory. It is stated, in the reference cited,[14] that some confirmation of these tests by field measurements has been obtained. Coefficients in the Weisbach formula were found to vary with the degree of rounding of contraction edges, the width of the openings, and the ratio of depths upstream and downstream from the contraction. The coefficients increased as the depths became more nearly equal. This variation supports the author's choice of a lower coefficient for the weir term.

One objection to the use of the Weisbach equation (or any other containing more than one term) is the complexity of the resulting equations for filling and emptying time. For the range of heads covered by Fig. 13—up to 6 ft—the weir term, although by no means negligible, is not large in comparison with the orifice term. Exponential approximations to the discharge equations may be used to simplify the integration and expedite calculation. Restricting the

[8] Associate Hydr. Engr., The Panama Canal, Balboa Heights, Canal Zone.

[8a] Received by the Secretary May 6, 1940.

[9] "Obstruction of Bridge Piers to the Flow of Water," by Floyd A. Nagler, *Transactions*, Am. Soc. C. E., Vol. LXXXII, December, 1918, p. 334.

[10] "Calculation of Flow in Open Channels," by Ivan E. Houk, M. Am. Soc. C. E., *Technical Reports*, Part IV, Miami Conservancy District (1918), p. 262.

[11] "Pile Trestles as Channel Obstructions," by D. L. Yarnell, *Technical Bulletin No. 429*, U. S. Dept. of Agriculture (1934).

[12] "Bridge Piers as Channel Obstructions," by D. L. Yarnell, *Technical Bulletin No. 442*, U. S. Dept. of Agriculture (1934).

[13] "Experiments on the Flow of Water through Contractions in an Open Channel," by E. W. Lane, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–1920), p. 1149.

[14] "Discharges through and over Chanoine Wicket Dams," *The Experiment Station Bulletin* (Hydraulics), U. S. Waterways Experiment Station, Vol. I, No. 3 (1938), p. 4 (condensed from a report of the U. S. Engineer Sub-Office (unpublished), Hydraulics Laboratory, Iowa City, Iowa, 1938.

argument to heads not exceeding 6.0 ft and values of d between 18 ft and 30 ft, Eqs. 3 are replaced, respectively, by the expressions

$$Q = 7.5\, b\, d\, h^{0.44} \qquad (12a)$$

for filling, and

$$Q = 7.75\, b\, d\, h^{0.53} \qquad (12b)$$

for emptying, the lock.

The differential equation $Q\, dt = -A\, dh$ may be integrated between the limits h_1 and h by inspection, yielding, as a result, expressions which replace the logarithmic Eq. 4*a* and the trigonometric Eq. 4*b*—namely,

$$T = \frac{1{,}450}{\sqrt{d}} \sqrt{h_1{}^{0.56} - h^{0.56}} \qquad (13a)$$

for filling, and

$$T = \frac{1{,}560}{\sqrt{d}} \sqrt{h_1{}^{0.47} - h^{0.47}} \qquad (13b)$$

for emptying, the lock. In the case which has been worked out by the author, Eqs. 13 reduce to forms which replace Eqs. 8 and 11, respectively,

$$t = 277 \sqrt{h_1{}^{0.56} - h^{0.56}} \qquad (14)$$

for filling and

$$t = 337 \sqrt{h_1{}^{0.47} - h^{0.47}} \qquad (15)$$

for emptying. These expressions can be solved very rapidly with a slide rule and give results for all purposes equivalent to the transcendental forms used by the author. To demonstrate this, Table 2 has been prepared from Eqs. 14 and 15 for comparison with Fig. 13.

TABLE 2.—FILLING AND EMPTYING TIMES, IN MINUTES

Head, in feet	Initial Head, in Feet; Filling						Initial Head, in Feet; Emptying					
	6.00	5.00	4.00	3.00	2.00	1.00	6.00	5.00	4.00	3.00	2.00	1.00
6.0	0.00						0.00					
5.0	2.40	0.00					2.45	0.00				
4.0	3.50	2.53	0.00				3.55	2.58	0.00			
3.0	4.33	3.60	2.57	0.00			4.50	3.76	2.76	0.00		
2.0	5.17	4.60	3.86	2.85	0.00		5.45	4.87	4.13	3.08	0.00	
1.0	6.10	5.58	5.00	4.27	3.17	0.00	6.47	5.95	5.40	4.63	3.46	0.00
0.5	6.60	6.20	5.60	5.00	4.10	2.56	7.14	6.70	6.13	5.50	4.57	2.98
0.0	7.62	7.25	6.80	6.28	5.58	4.61	8.53	8.20	7.80	7.30	6.60	5.62

In comparing Table 2 with Fig. 13, it must be kept in mind that a 9-in. total width of openings represents 1 min of time. Calculations for the table were made with a slide rule and are not necessarily correct in the third significant figure.

Aside from obvious advantages in calculation, particularly for preliminary designs, it is believed that inspection of Eqs. 14 and 15 gives a better idea of the filling and emptying operations than does examination of the transcendental forms, despite the empirical basis of the former. It should be emphasized, however, that the use of these exponential approximations is merely an ana-

lytical trick, possessing no significance whatever beyond the study of the case at hand.

In conclusion, a word of commendation for the good comparison between the author's theory and the test operations is in order. The agreement is indeed excellent. It appears that no allowance was made for flow under the gates when they were not in contact with the sill against which they sealed. In some cases, particularly with an elevated sill, this flow might be quite considerable. It would be interesting to know if this flow was negligible or if the coefficients in the discharge equations were chosen a little high to allow for it.

W. C. Weeks,[15] M. Am. Soc. C. E. (by letter).[15a]—The description and illustration of the design and construction of the new navigation lock in the Chicago River present a masterpiece of lucid engineering presentation. Building a durable and serviceable structure of this magnitude within a reasonable time, and, more particularly, for materially less than the public funds appropriated therefor, is not common in recent years. The accomplishment reflects unusual and well-deserved credit upon the engineers of The Sanitary District of Chicago and their assistants.

The new lock is intended primarily as a safeguard to the water supply of the City of Chicago. For this purpose it appears to be admirably adapted. Incidentally, its construction, and that of essential appurtenant structures, provides for positive control of the diversion from Lake Michigan, maintenance of the level of the Chicago River below that of the lake, and desirable anchorage areas. With respect to navigation, however, the lock appears to be an impediment, inasmuch as every craft entering or leaving the river must use the lock. This involves a delay of at least 15 min for each lockage.

Mr. Ramey's paper doubtless has been studied in connection with the one by Major Karrick,[2] who recounts, in comprehensive manner, the difficulty of the city in maintaining the purity of its water supply ever since it had one, and that of The Sanitary District to the same end. With respect to the prevention of lake pollution, mention is made of suggested methods other than a navigation lock. None of these, however, were acceptable to all concerned, but the objections were not specifically set forth.

One of the suggested methods which seems to merit special mention contemplated closure of the river during intervals of flow reversal by means of a pontoon gate hinged at one abutment. This plan, if adopted, apparently would have solved the problem with respect to lake pollution. It would have afforded a navigable pass 220 ft wide instead of an 80-ft lock chamber. Probably its interference with navigation in general between lake and river ultimately would be much less than that due to lockage. Finally, its cost would be much less than that of a lock.

From the record, with a diversion of 5,000 cu ft per sec, the pontoon would be closed from 10 to 12 times per year. Such closures would prevail over maxi-

[15] Col., Corps of Engrs., U. S. Army (*Retired*), Bridgeport, Conn.

[15a] Received by the Secretary May 11, 1940.

[2] "Protecting Chicago's Water Supply," by Samuel N. Karrick, *Civil Engineering*, September 1939, p. 547.

mum periods of 9 hr; the average probably would not exceed 7 hr. Chicago city bridges are not operated for navigation during rush traffic hours. Closed bridges are an absolute bar twice daily to such craft as cannot pass beneath them. Relatively few lake vessels use the river, and the bulk of navigation consists of tugs and barges. For this purpose, the pontoon gate appears to offer material advantages. Interchange of lake-borne and river-borne cargoes over the North Pier, if developed for such purpose, will reduce the number of lockages. However, civic attitude toward commercialization of the lake front probably will minimize, and may entirely prevent, such development.

Among the advantages claimed for the lock is the ability to lower the level of the Chicago River. This provides greater clearances under the bridges, a reservoir for moderate runoffs, and facilitates its prompt discharge. Lowering the river level is not necessarily limited to 2 ft below lake level as now contemplated. Demands for additional clearances may arise. If granted, the stability of many structures adjacent to the river would be endangered seriously. Installation of a pontoon gate would have forestalled such demands.

Traffic on the upper reaches of the Lakes to the Gulf Waterway is borne on the raw sewage of a city of a million people. The volume of this sewage two thirds of the time is 2,700 cu ft per sec. The maximum current is about 0.67 ft per sec; that in the Chicago River is almost negligible; and in the pools above the locks it is just about nil. From a sanitary standpoint, these are not navigable waters.

Obviously, the less the diversion from Lake Michigan, the more often the pontoon gate would be closed and the greater its hindrance to navigation. At hearings before a special master, the late General Edgar Jadwin, M. Am. Soc. C. E., then chief of engineers, testified in effect that a diversion of 5,000 cu ft per sec from Lake Michigan would be necessary to safeguard navigation on the Chicago River. It is believed that the soundness of this testimony will be verified during the observation period now presumably in effect. If so, a pontoon gate with respect to Chicago River navigation appears advantageous.

However, the new lock is a reality, apparently satisfactory to all concerned, and for this reason should be accepted as the best solution for the problems involved. With respect to cost, it is an unusual accomplishment. Ingenious modification and adaptation of standard types of construction effected material savings with no sacrifice of permanence, stability, or usefulness. Particularly noteworthy is the design of the lock gates for the special conditions to be met. Taken in its entirety, it is an example of genuine engineering. The exercise of careful thought and planning is evident throughout. To effect so material a saving in the expenditure of public funds nowadays is conspicuous for its rarity.

It is believed that the engineering profession, particularly with respect to the younger members engaged on "projects" financed by public money, has suffered serious deterioration in recent years. Many of these projects have been politically conceived, located, and constructed. Economic justification in very many cases has not been established. It also is believed that the technical press has not fulfilled its mission adequately with respect to publicly financed engineering undertakings. A fearless and forthright presentation of

such undertakings from an economic standpoint would have been of great educational value. Such presentations rarely have been made. The general public should not have to depend for its enlightenment upon nontechnical writers in current magazines as to how, where, and why its money is spent.

Mr. Ramey's contribution to the profession will be sincerely—even gratefully—appreciated by all engineers who believe in the homely old adage that "An engineer is a man who can do with one dollar what any fool can do with two." To such, it is as an oasis in a desert.

HENRY R. KING,[16] ESQ. (by letter).[16a]—An unusually novel feature of the Chicago River lock is the triangular-shaped gate leaves, providing three passages for the admission of the water when the gates are in a partly opened position, in place of one central passage if true sector gates had been used. Separation of the inflow into three streams tends to dissipate the kinetic energy of the incoming water with reduced turbulence within the lock. This is particularly true if, as in the case of the Chicago lock, the two streams entering the gate recesses are deflected back toward each other impinging on opposite sides of the stream flowing through the central passage. The suitability of the gates for low head locks was convincingly demonstrated in the test of June 25, 1938, when the lock was filled from an initial head differential of 8 ft. Very little turbulence occurred in the lock during the filling operation.

The effect of various factors on the time required for operation of the lock may be more apparent if Eqs. 4*a* and 4*b* are changed to the form for filling:

$$T = \frac{5{,}480}{C^{0.5}\,R^{0.5}\,d^{0.25}}\left[\log\frac{1.31\,d^{0.5}+h_1^{0.5}}{1.31\,d^{0.5}-h_1^{0.5}} - \log\frac{1.31\,d^{0.5}+h^{0.5}}{1.31\,d^{0.5}-h^{0.5}}\right]^{0.5} \quad ..(16a)$$

and for emptying:

$$T = \frac{5{,}550}{C^{0.5}\,R^{0.5}\,d^{0.25}}\left[\tan^{-1}\left(\frac{h_1}{2.40\,d}\right)^{0.5} - \tan^{-1}\left(\frac{h}{2.40\,d}\right)^{0.5}\right]^{0.5} \quad ..(16b)$$

in which R = the rate of increase in the combined width of the three openings, in inches per minute; C = the coefficient of contraction; and the other symbols are as indicated by the author.

During a filling or an emptying operation the discharge at any particular head is proportional to the total width of the three openings, which varies as $R\,T$. The time, however, varies inversely as $R^{0.5}$, and therefore the discharge must be proportional to $R^{0.5}$. For example, in Fig. 13 the maximum rate of discharge into the lock when filling from El. − 3.0 to + 3.0 is 1,120 cu ft per sec. If the opening rate at low speed were increased four times, the maximum rate of discharge into the lock would be $4^{0.5} \times 1{,}120 = 2{,}240$ cu ft per sec, and the time required for equalization of levels would be $\frac{1}{4^{0.5}} = 0.5$ths as long.

The test of June 25, 1938, was a good check of the accuracy of the formula for computing the filling time. Since the time varies inversely as the square

[16] Senior Civ. Engr., The Sanitary Dist. of Chicago, Chicago, Ill.

[16a] Received by the Secretary May 17, 1940.

root of the contraction coefficient a reasonable error in its selection would only slightly alter the results.

The author's formula for discharge into the lock may be derived in another way if it is assumed that a horizontal submerged plate at the elevation of the water surface in the lock extends from the openings out into the lake. Then the water entering from above this plate would flow through the openings at critical depth and that from below would be the true orifice discharge, and the formula would be

$$Q = C\left[\frac{2}{3}b\,h\left(2\,g\,\frac{h}{3}\right)^{0.5} + b(d-h)(2\,g\,h)^{0.5}\right]\ldots\ldots\ldots(17)$$

Letting $C = 0.95$, this reduces to

$$Q = 7.62\,d\,b\,h^{0.5} - 4.69\,b\,d^{1.5}\ldots\ldots\ldots\ldots\ldots\ldots(18)$$

which corresponds to Eq. 3*a*; and for normal operating conditions the results will not differ from the author's value by more than 0.5%.

From the standpoint of turbulence in the lock the kinetic energy of the incoming water is of more importance than the rate of inflow. This is expressed as $\frac{W\,v^2}{2\,g}$ in which W = weight in pounds. The kinetic energy of the discharge of Eq. 17 is as follows:

$$K = C\left[\frac{2}{3}b\,h\left(2\,g\,\frac{h}{3}\right)^{0.5} \times 62.5\,\frac{h}{3} + b(d-h)(2\,g\,h)^{0.5} \times 62.5\,h\right]\ldots(19)$$

in which K = kinetic energy of the inflow per second, in foot-pounds. Simplified, the formula becomes

$$K = C\,[501\,d\,b\,h^{1.5} - 437\,b\,h^{2.5}]\ldots\ldots\ldots\ldots\ldots(20)$$

For the 6-ft fill from El. − 3.0 to + 3.0 (see Fig. 13) the maximum kinetic energy of the inflow occurs at a discharge rate of about 960 cu ft per sec and is 239,000 ft-lb per sec. The kinetic energy at the maximum rate of inflow of 1,120 cu ft per sec is only about 73% as great, or 173,000 ft-lb per sec.

Triangular-shaped gates have certain advantages, particularly in entering the water into the lock in three streams which result in more effective dissipation of its kinetic energy. However, the flow of water into the gate recesses carries with it debris and, in cold weather, ice, which may lodge there and interfere with gate operation. Inasmuch as these were problems to be worked out from operating experience it will be of interest if the author discusses just what modifications, if any, are now contemplated in the gates and just what difficulties have been encountered in this respect.

Corrections for *Transactions:* The first bracket in Eq. 8 should include "log." In the legend to Fig. 14, the theoretical line should be dashed rather than solid; and on page 101, line 4, change "1.5" to "0.65." See also June *Proceedings*, footnote 6*c*.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

MEASURING THE POTENTIAL TRAFFIC OF A PROPOSED VEHICULAR CROSSING

Discussion

By Murray D. Van Wagoner, M. Am. Soc. C. E.

Murray D. Van Wagoner,[5] M. Am. Soc. C. E. (by letter).[5a]—To attempt the solution of any great problem, whether it be in science, economics, politics, engineering, or the art of war, one must have a plan of attack. The paper by Mr. Cherniack furnishes the engineer with an excellent outline of attack upon the problem of determining the economic feasibility of a proposed vehicular crossing. The major difficulties to be encountered are mentioned and ways of overcoming them suggested.

The first step is to determine the boundaries of the territory from which the traffic for the new crossing will come. Then the amount of traffic in this "reservoir" must be determined. To determine this volume, short-time traffic counts must be made at the proper time of day, week, and year. It appears that the 12-hr period from 7 a.m. to 7 p.m. on Wednesdays or Thursdays, or both, with a supplementary count on Sundays in the months of April, May, September, or October (or in all four months), is the best time for such counts.

Knowing the contents of the traffic "reservoir," the portion of it likely to use a crossing such as the one proposed must be determined. This will be difficult and may be very expensive, but it is the most crucial point in arriving at a financially successful solution to the problem; in fact, it is the problem. Regardless of the fact that the traffic of the new crossing may be expected to come from both existing and new traffic, the existing portion is all that is available for study; and estimates of the portion to come from each must be based on what now exists.

A study of diversion experience factors of existing toll crossings shows such wide variation that only general conclusions can be drawn from them. They do indicate that the largest portions of the traffic of a new crossing will come from crossings nearest it and from those with "the smallest favorable toll

Note.—This paper by N. Cherniack, Assoc. M. Am. Soc. C. E., was published in February, 1940, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[5] State Highway Commr., State Highway Dept., Lansing, Mich.

[5a] Received by the Secretary March 15, 1940.

spreads." To measure probable diversion accurately, origin-and-destination studies should be made and "lines of travel" established.

Mr. Cherniack gives a very novel method for estimating the probable diversion. For each competitive crossing a "relative merit rating" is established by giving consideration to waiting time, running time, distance, and amount of toll for the various "lines of travel" using it. Monetary value is assigned to these items and the final rating of each existing crossing determined by using this cost factor and the present volume of traffic carried. Then to determine the rating of the proposed crossing another equation is given. With the ratings of the existing crossings and that of the proposed crossing, the amount of divertible traffic is readily calculated. All this can be illustrated by simple mathematical equations.

A possible weak spot in the paper may be found in the equation for determining the rating of the proposed crossing. Mr. Cherniack does not explain just why this should be in the nature of a compound interest or discount formula; nor does he explain how the value of d, the amount of discount, is to be determined. It is merely defined as the "discount factor by which unity (1) is reduced for every cent of excess travel cost over that via the 'standard' crossing." He does not make the statement that the rating of the proposed crossing was obtained by using judgment and the equations. Now if one's judgment is to enter into the determination of a factor or term in a mathematical equation, the equation becomes merely a group of symbols representing an idea and ceases to be a mathematical tool.

It was found that comparative cost and comparative patronage of several existing crossings were related, but there was evidence of still other factors influencing the use of one crossing in preference to another. The conclusion to this was that there were certain things not easily measured in dollars and cents, such as scenery, safety, travel habits, etc., which play a part in a motorist's decision of which crossing he will use.

In the event that the divertible traffic appears insufficient to warrant the construction of a new crossing, one can (1) wait until it is sufficient, or (2) anticipate expansion. The first choice may result in abandoning construction of the crossing. The argument is then presented that if the new crossing is to be built merely to accommodate present traffic more advantageously, and no expansion is in sight, there is little use of building it at all. If the engineer builds with an eye not only to diverting some of the existing traffic to the new crossing but to future expansion of traffic, he must try to estimate the amount of this expected traffic.

There are all kinds of methods for estimating future traffic. A large variety of mathematical formulas has been used on such things as vehicle registration, population, gasoline consumption, etc., to estimate future traffic. All these have their statistical pitfalls. If the new crossing can add greatly to convenience of travel, new traffic will be generated automatically in addition to that which comes about by normal expansion.

Other structures fundamental to the success of a proposed toll crossing are adequate feeders, approaches, and plazas. Not only must these be adequate, but they should be superior to those servicing the competitive crossings; and,

if they are not, they should be made so, at the same time taking care that they do not also aid competitive crossings. It is interesting to note the increase in traffic through the Lincoln Tunnel as various streets leading to it are improved.

It may also be necessary to conduct advertising campaigns to increase patronage. According to past experience, the lowering of tolls does not usually increase revenue through increased traffic.

The importance of checking traffic estimates must not be overlooked. It is quite possible that between the time of the original estimates and the completion of the new crossing radical changes such as economic depressions, increased facilities for feeding competitive crossings, toll rates, etc., may take place and these materially affect the traffic over the new crossing.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SEALING THE LAGOON LINING AT TREASURE ISLAND WITH SALT

Discussion

By Messrs. G. B. Bodman, and John D. Watson

G. B. Bodman,[7] Esq. (by letter).[7a]—In the early part of his paper Mr. Lee mentioned two methods of rendering clay beds impervious. The first of these is chiefly mechanical and depends upon so-called puddling processes which produce partial dispersion and compaction. The second method is chiefly chemical and depends upon the application of the physicochemical principles of colloidal chemistry.

Those who are interested in soils because of their agricultural importance are naturally concerned with the fundamental properties of their constituents, not only because of their immediate rôle in plant nutrition, but also because of the somewhat less direct effects they may exert upon plant growth and crop production by way of the physical properties which they impart to the soil mass.

Interest in the physical properties of soils is greatly stimulated, in the west, by a phenomenon which is almost exactly the opposite of that which demands the attention of the engineer. Whereas clay beds, or artificially distributed layers of clay soils, may tend to possess too low a density, too high a porosity, and too great a permeability for the purpose of the engineer, the experience under irrigation agriculture in Western United States is that certain soils may change under irrigation, and tend to become too dense and to assume too low a porosity when cultivated. Furthermore their permeability to, and ease of penetration by, irrigation water may be so greatly reduced that their management is much complicated and their productivity likely to be seriously impaired. The agriculturist must frequently seek, therefore, to overcome the very conditions which the engineer may desire to create.

For these reasons Mr. Lee's experience with the Exposition Lagoon is of special interest to the soil scientist since a condition has been produced which is beneficial from the engineer's viewpoint but which from the point of view of the farmer is harmful and similar to that which may be produced in soils,

Note.—This paper by Charles H. Lee, M. Am. Soc. C. E., was published in February, 1940, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[7] Prof. of Soil Physics, Coll. of Agriculture, Univ. of California, Berkeley, Calif.

[7a] Received by the Secretary March 8, 1940.

either in field or laboratory, by the continued use of saline irrigation water high in sodium.

Knowledge of the constitution of mineral clays, such as those now known to occur in soils, is being extended almost daily by workers in various parts of the world. Accordingly the diagrammatic models which have been designed for aid in the interpretation and prediction of the behavior of clays require frequent revision. Certain relatively conservative and simple pictures of the mechanisms may still be utilized, nevertheless, and actually prove most useful in visualizing the behavior of clays under given conditions. It should be mentioned, however, that much of the existing physicochemical information depends upon experience with suspensions of clay. Much less really fundamental work has been performed upon pasty masses of clay in which the ratio of solids to liquids is relatively high.

The clay particles in soils may be considered as particles of colloid each one of which acts like a weakly dissociated molecule of polybasic aluminosilicic acid possessing a high molecular weight. In addition to releasing more or less hydrogen ions when suspended in, or bathed by, water, they also release, in varying proportions, cations of the metals commonly found in soils—namely, calcium, magnesium, sodium, and potassium. These may be replaced by electrically equivalent amounts of other ions and are said, therefore, to be exchangeable. The kind and degree of dissociation are such that the resultant system, while electrically neutral as a whole, may be visualized as possessing a bulky, negatively charged core (the anion) surrounded by a swarm of much smaller, positively charged bodies (the cations). The cations are in a state of continual movement about the anion and are, on the average, more numerous near its surface than they are farther away.

The magnitude of the negative charge on the particle and also the mean distance between the particle and the concentric shell of cations appear to be measures of the permanence with which a water suspension of clay particles can resist flocculation. The magnitude of the negative charge depends upon the chemical constitution of the particle. The amount of dissociation and the mean distance between negative charges on the particle and the center of gravity of the positive charges which surround it depend upon (1) the kinds of cations present—sodium, if present, being most highly dissociated from the particle, hydrogen being least dissociated, and potassium, magnesium, and calcium, in decreasing order, being intermediate in dissociation; and (2) the abundance and kind of electrolytes, or salts, present in the surrounding water.

The same factors that control the stability of a clay suspension (that is, its resistance to flocculation and to the tendency for its particles to cohere upon impact into clumps or aggregates of considerable bulk and to settle out into a loosely packed mass of high void ratio) appear to exert a profound effect upon the behavior of clay particles in a body of soil. When sodium predominates as the exchangeable cation, the clay particles in a mass of soil impart to it physical properties which are conspicuously clayey. The reverse is generally true if calcium is the predominating exchangeable cation. Since an ionic exchange (termed base exchange) may be effected by sheer force of ionic numbers, soil containing a large number of calcium ions in association

with its clay particles may have them exchanged for sodium ions by a sufficiently long leaching with water containing a sufficient number of dissociated sodium ions. The leaching serves the twofold purpose of supplying the sodium ions and removing the displaced calcium ions. As long as the soil pores are occupied with salt-rich water, the exchange of calcium for sodium may produce relatively little effect upon the physical behavior of the soil mass. Upon removal of the excess salts by washing, however, the newly formed sodium clay will dominate the behavior of the soil mass; the aggregates will tend to disperse; the streaming water will cause local migration of the liberated individual clay particles, blocking the finer pores; and the entire mass will become sticky, retentive of water, and highly impermeable. A partial reversal of these conditions may be brought about by replacement with calcium although the cycle of alterations is not strictly reversible.

Utilization of the physicochemical method of reducing the permeability of a soil mass requires the consideration of the following points: (1) The nature

TABLE 9.—Permeability of Yolo Silty Clay Loam as Affected by Proportion and Concentration of Sodium and Calcium Chlorides in the Percolating Water During Both the Preliminary and Reclamation Treatments

Composition of Water During Preliminary Treatment					Permeability (Water-Saturated), as Inches per Day per One Gravity; During Tenth Day of:							
Concentration[a]	Amounts Present per Liter, of:				Preliminary Treatment with Salt Water				Reclamation with Distilled Water			
	Sodium		Calcium									
In parts per million	Grams	Milligram equivalents	Grams	Milligram equivalents	a	b	c	Average	a	b	c	Average
(a) High Calcium Water (Ratio of Sodium to Calcium, by Equivalents, Equals 1 to 8)												
5000	0.229	9.95	1.595	79.6	13.3	12.8	10.0	12.0	1.2	1.3	1.0	1.2
500	0.023	0.995	0.159	7.96	7.6	7.5	9.5	8.2	0.77	0.85	0.95	0.86
(b) High Sodium Water (Ratio of Sodium to Calcium, by Equivalents, Equals 8 to 1)												
5000	1.758	76.5	0.192	9.56	5.0	6.0	6.5	5.8	0.03	0.03	0.03	0.03
500	0.176	7.65	0.019	0.956	4.4	1.7	2.9	3.0	0.27	0.21	0.19	0.22

[a] Concentration is given in parts of sodium chloride plus calcium chloride per million of water. Thus 5,000 ppm indicates, in the case of high calcium water, that 1 liter of such water contains 5 g of the mixed chlorides in the proportions—0.229 g sodium : 1.595 g calcium : 3.176 g chloride. The less concentrated salt waters having only 500 parts of mixed chlorides per million of water contain, respectively, the same materials in the same proportions but in one tenth the amounts.

of the soil, its composition, and existing base status; (2) the removal of excess electrolytes; and (3) if the desired cation is added in solution, the composition of the water used. Tables 9 and 10 will help to illustrate the significance of these considerations.[8]

[8] From data obtained in the laboratory of the Div. of Soil Technology, Univ. of California, by G. B. Bodman and M. Fireman.

Referring to Table 9, the soil columns were first saturated with distilled water from below, and the saline water was then added from above for downward percolation. Before reclamation began with distilled water, flow was interrupted for 17 days, the soil remaining in contact with the saline water

TABLE 10.—PERMEABILITY OF TWO REPRESENTATIVE SOIL TYPES AS AFFECTED BY PRELIMINARY TREATMENT WITH HIGH SODIUM WATER

Description	Yolo silty clay loam		Aiken clay loam	
Composition of Water Used During Preliminary Treatment:				
Ratio of milliequivalents of sodium to calcium	—	6 : 1	—	6 : 1
Concentration of sodium chloride plus calcium chloride, in parts per million	0	4,000	0	4,000
Permeability (Water-Saturated), as Inches per Day per One Gravity:				
During the 46th day of preliminary treatment	0.31	0.75	7.3	21
During the 10th day of "reclamation" with water containing 500 ppm of chlorides (Na : Ca = 6 : 1)	0.30	0.64	6.1	16

during that interval, until the distilled water was added from above, for downward flow. Trials were made in a basement laboratory at an average temperature of 26° C.

The Yolo soil has a rather high clay and silt content and originally is very low in exchangeable sodium. Note that ten days of leaching with more or less saline irrigation water results in a set of permeability values which have magnitudes associated with the abundance of sodium in the water. The water containing 5,000 ppm of calcium and sodium chlorides in the ratio of eight equivalents of calcium to one of sodium tends to maintain a high permeability. The calcium status of the soil is increased and there is relatively little tendency to disperse. Reduction in salt concentration, however, even if the ratio of calcium to sodium remains high, produces a relatively lower permeability, apparently due to the increased dispersibility and reduced mean effective pore radii within the soil which result from a smaller electrolytic content in the water.

The effect of the sodium-rich waters is distinct—still lower permeabilities are produced; but here again, although both waters have the same sodium-to-calcium ratio, the salt-concentration effect—due to repressed dissociation of the soil clays—is more pronounced in the water containing 5,000 ppm than in that containing only 500 ppm.

Upon examination of the conditions during the ensuing washing process with distilled water, it was found that the permeability was reduced to one tenth to one two-hundredth of its former value by removal of the excess salts. The sealing effect produced by the preliminary treatment with water containing 5,000 ppm of sodium chloride plus calcium chloride in the equivalent ratio of Na : Ca = 8 : 1 is most pronounced of all, and after 10 days of washing the column receiving this treatment had reached the low value of 0.03 in. per day under a driving force of one gravity. The entire series of washed soils possessed permeabilities in a predictable sequence. Chemical analyses of the leached soils from these particular trials have not been made; but results with other,

similar experiments justify the expectation that, during the time period concerned, the high calcium water has produced a more calcium-rich soil at the expense of magnesium and sodium, and that the high sodium water has increased the exchangeable sodium in the soil about 500% to 1,000%, mainly at the expense of the magnesium. Complete replacement of existing cations by sodium has not been obtained under these conditions of leaching. Furthermore, the extent of alteration doubtless has been in proportion to the concentration of salts in the two chemical types of water.

Table 10 is presented primarily to illustrate the difference in behavior exhibited by two soils of rather similar apparent mechanical composition but very different clay constitution. The trials were made in a room maintained at a constant temperature of 30° C. The first, the Yolo silty clay loam, contains a highly siliceous clay of the montmorillonitic (bentonitic) type. The second, the Aiken clay loam, contains a much more ferruginous and aluminous clay of the kaolinitic type.

The former is potentially a much more favorable material than the latter from the engineer's point of view, in so far as providing an impermeable clay lining is concerned. The Aiken responds extremely poorly to any attempts to disperse it and render it retentive of, and impermeable to, water. Table 10 also illustrates the importance (if impermeability is desired) of removing excess salts by means of a relatively pure water. Note that the so-called "reclamation" process here consisted of washing with a water containing 500 ppm of high sodium water. The use of salt-free water would have reduced the permeability of the Yolo much more rapidly and completely, although as judged by other experiments not here reported it is extremely doubtful whether the Aiken, owing to the nature of its clay, could ever be reduced to the high degree of impermeability exhibited by the Yolo.

The Soil Technology Laboratory has been able to produce very low permeabilities in artificially packed columns of several different soils by long-continued leaching with pure water alone. The explanation seems to lie in the removal of the excess salts (originally present as the soil solution), dispersion, and subsequent local migration of clay, and in the marked reduction in the number of the coarser micropores in the soil. Hitherto unexplained increases in permeability, however, both to salt-free and saline water have repeatedly been obtained in the Soil Technology Laboratory, University of California, during long-continued percolation of water columns.

Finally, it should be emphasized that the entire permeability-time curve for a clay seal in a water reservoir, not only during its preparatory treatment by leaching, but also later during its actual use, will depend upon numerous factors. The permeability at any given time is partly controlled by the permeability up to that time, since it is the opportunity of exchange between ions at the surfaces of the individual clay particles and ions held in solution by the percolating water which is the essential preliminary to any alteration in base status of the clay and hence also to alterations in its physical behavior. The clay particles contained in the seal always tend to come to physicochemical equilibrium with the water solution by which they are bathed, so that an equilibrium distribution of ions between clay and liquid is approached, if not

actually attained. Evidently after preparation of the seal, replacement with sodium, washing to remove electrolytes, and filling the reservoir with storage water, a fresh approach will be made toward a new physicochemical equilibrium between clay and solution (the storage water). Long-time storage of high-lime water, or of highly saline water, would not prove successful, and it seems probable that, even with water of a high degree of chemical purity, an occasional replenishment of sodium ions in the clay seal might be desirable at intervals. So far no experiments have been conducted continuously over a sufficiently long time to answer this latter question.

John D. Watson,[9] Assoc. M. Am. Soc. C. E. (by letter).[9a]—The author states (see heading "Conclusions") that "this method of sealing would appear to have a wide field of usefulness in constructing impervious membranes for water works structures such as reservoirs and dams." Although the writer cannot deny the efficacy of the salt in ionizing the solution and thereby hastening the effectiveness of the clay seal, nevertheless it must be pointed out that if the clay had been properly applied, the salt would not have been needed.

In the first place it was an error to buy a soil to be used for sealing the Lagoon on the basis of the particle sizes of the soil. The permeability of a soil can be indicated only very approximately by a mechanical analysis. The late Allen Hazen, M. Am. Soc. C. E., made a point[10] of this fact in 1924, but many engineers are still beguiled by the imagined magic of the grain size distribution curve. The best measure of imperviousness is the coefficient of permeability according to Darcy's law, and this coefficient can be determined very readily by a soils laboratory. The most desirable material to use in this case would have been the material with the lowest coefficient of permeability, provided it was not subject to excessive swelling which would cause an increase in its permeability. Other tests by the soils laboratory could have measured this factor of swelling.

After it has been dried, clay will slake when it is wetted again. The layer of clay in this Lagoon was subjected to two months of severe drying, and the author states that the entire layer was fissured with shrinkage cracks. It is no surprize, therefore, that when water was put into the Lagoon, the leakage was excessive and "the clay was found to have softened to a mush for a depth of 8 in. or more." Since the clay was allowed to dry out completely and was then wetted again, it slaked. This slaking accounts fully for the high rate of seepage and for the complete softening of the clay layer. The tests to determine the optimum moisture content, the sprinkling process, and the rolling were all so much wasted effort after the clay was allowed to dry out. Incidentally, why were the tests to determine optimum moisture content ever made at all, since "no effort was made to attain a condition of uniform moisture before rolling"?

The author points out that when salt water was put into the Lagoon "the clay, although softening to a depth of 2 in., was hard and cohesive for the

---

[9] Asst. Prof., Civ. Eng., Coll. of Eng., Duke Univ., Durham, N. C.

[9a] Received by the Secretary March 21, 1940.

[10] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 62.

remainder of its depth." The previously mentioned 8-in. depth of softening was with fresh water. The author does not state with equal emphasis that the 8-in. depth of softening was determined six days after fresh water was admitted to the Lagoon, whereas the 2-in. depth of softening was determined not less than sixty-five days after sea water was put in. The writer ventures the opinion that at the end of the first six days the clay was softened by the salt water to the same depth to which it had been softened by the fresh water. During this time interval of two months the clay was reconsolidated by a force which the author has not mentioned. Karl Terzaghi, M. Am. Soc. C. E., has called this force, produced by any fluid flowing through a porous medium, the "seepage pressure." It is simple to show by elementary mechanics that the magnitude of seepage pressure is equal to the product of the unit weight of the fluid, the hydraulic gradient, and the volume of the medium. With an 18-in. depth of sea water over a 10-in. layer of clay, the seepage pressure amounted to 150 lb per sq ft. Although this amount is insignificantly small when compared with any type of roller, it is nevertheless large enough to consolidate a mass of very soupy clay to a high degree of impermeability after a sufficient period of time. In this particular case 100% consolidation might have been reached in two months, although it is most unlikely that it could have been reached in six days.

The points which the writer has endeavored to make are these: (1) The clay should have been judged on the basis of its permeability rather than the sizes of its particles; (2) the clay should have been submerged before it was allowed to dry; and (3) even though the clay was allowed to dry, ultimately it would have sealed itself without the salt, although the salt was advantageous in hastening the process.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FLOOD-CONTROL METHODS; THEIR PHYSICAL AND ECONOMIC LIMITATIONS

PROGRESS REPORT OF COMMITTEE OF THE HYDRAULICS DIVISION ON FLOOD CONTROL

Discussion

BY MESSRS. CHARLES H. PAUL, W. C. HAMMATT, AND WILLIAM P. CREAGER

CHARLES H. PAUL,[10] M. AM. SOC. C. E. (by letter).[10a]—This Report is timely, well prepared, and worthy of careful study by any one concerned with the initiation, promotion, or approval of flood-control projects. The "Introduction," alone, covers the general situation in a way that impresses the reader at once with the logical approach the Committee has made to the subject.

It is difficult for a reader to select particular features of this Report for discussion, for all of them are important, and special emphasis on one feature might seem to detract from the importance of the others. It is all good, from the timely reference to limitations, in the early part of the paper, to the sound basis for economic justification expressed in the last paragraph.

The paragraph on "Flood Records" (paragraph 12), emphasizes and illustrates the value and necessity of reliable and continuous flood records. Reference might be made here to the U. S. Geological Survey *Water Supply Paper No. 771*, "Floods in the United States. Magnitude and Frequency," which is the result of an effort sponsored in 1934–1936 by the Mississippi Valley Committee of the Public Works Administration (PWA). This work in turn took advantage of the prior effort along this line initiated by the Special Committee on Flood Protection Data which was active in the years 1922 to 1928, and which "performed a very important preliminary work in selecting such records as would be of most practical use, such forms as would be most helpful for their presentation, and such methods of analysis as would elicit the desired generalized information."[11]

NOTE.—The Progress Report of the Committee on Flood Control was published in February, 1940, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: May, 1940, by Lynn Crandall, M. Am. Soc. C. E.

[10] Cons. Engr., Dayton, Ohio.

[10a] Received by the Secretary April 22, 1940.

[11] "Floods in the United States. Magnitude and Frequency," U. S. Geological Survey *Water Supply Paper No. 771*, p. 16.

In general, the tabulation of floods in *Water Supply Paper No. 771* is limited to measurement stations with reasonably continuous records of 20 years or longer. It was the hope of those interested in that publication that it would be followed, at least every five years, by supplements which would bring the tabulations up to date and include other important records which would then be qualified under the 20-year limit. That procedure would greatly enhance the value of these flood tabulations, not only by introducing additional records, but also by continually bringing the older ones forward to cover longer and longer periods. As a much needed basis for purposes of flood control studies, a more important project can scarcely be conceived.

The material included in *Water Supply Paper No. 771* is now (1940) practically five years old. It is hoped that the Committee will take an active and aggressive part in promoting the preparation and publication of a (first) supplement through cooperation, of course, with the U. S. Geological Survey, and in getting this work established as a recognized part of the routine procedure of that department of the government.

W. C. Hammatt,[12] M. Am. Soc. C. E. (by letter).[12a]—The members of the Committee are to be congratulated on the comprehensive, concise, and instructive statement of the general and specific principles governing the control of hazardous flood conditions on major streams. Their outline of this important subject, while advancing little with which the engineering specialist in that line is not familiar, places the basic principles in a logical sequence readily understandable by the layman and by the engineer whose experience has been more general. Their presentation of the major principles of the practical and economic features of flood control is so thorough that there is little to add. However, the writer wishes to expand and clarify a few of the points made in the Report.

In paragraph 46 it is stated: "The enlargement of the channel of a large river by widening or deepening is seldom a practicable method of flood relief, except for short distances for the protection of cities." The writer wishes particularly to call attention to the program for the control of the Los Angeles River, now, and for some years past, in progress of development by the U. S. Corps of Engineers. This program, started as a channel protection through the city proper, has been extended to include practically the entire length of the river and at least one of its major tributaries. Without going into the history and regimen of the stream or a descriptive outline of the protective works (which would be of sufficient scope and interest to furnish material for an entire paper), it may be said that the initial construction work in channel improvement, by aggravating conditions above and below in the major flood following its completion, demonstrated that the improvement, to be effective, must include the entire channel. The writer offers this as an illustration that local channel improvements, by changing the regimen from that which Nature has built up, cause the river to strive to return to its original condition. The authors have made note of this fact in paragraph 48 of the Report.

[12] Civ. and Cons. Engr., Los Angeles, Calif.

[12a] Received by the Secretary April 22, 1940.

Particular reference is made to the advocacy of shortening the channel by means of cutoffs. Most rivers have been formed under natural physical laws, and any attempt to go contrary to these laws causes Nature to fight back. Wherever, therefore, the engineer proposes to interfere with the natural regimen of a stream, he must count the cost of combating Nature and evaluate his chances of success in the fight. One of the major tendencies of a natural stream is to make and preserve a uniform grade. In the study of many rivers it has become apparent that, unless influenced by a natural "control," the tendency is toward a uniform grade, gradually flattening from the head to the outlet. In general, local meanderings are Nature's method of adjusting this uniformity of grade. Experience has shown that the making of a cutoff across a bend generally causes the filling of the channel below the new cut and the formation of a bar, in some cases, ultimately inducing the river to return to its old course, but almost invariably causing trouble on the stretch below the cutoff.

Channel improvements may be major or minor. The writer's experience has been that major improvements, if not carried to complete canalization, as in the case of the Los Angeles River, are a constant hazard and expense. Minor improvements, often effective where great property values are not involved, consist in coaxing Nature to do its own work. Such improvements consist of influencing the trend of the channel by means of current deflectors or retarders, whereby the deposit of silt is induced and a new channel formed in a selected location.

Another matter worthy of more complete treatment is that of by-pass channels, whereby the flow in excess of the carrying capacity of the main channel may be carried by an auxiliary route, being admitted thereto either by controlled inlets or by fuse-plug levees. It is a characteristic of silt-bearing streams in semiarid climates (that is, where there is a marked seasonal variation in the stream flow) that under natural conditions their beds are built up in the season of receding flow, causing overtopping and bank deposits at time of high flow, thus, through the years, building up a ridge along which the river flows, flanked by depressed areas. Under natural conditions, an excessive flood may cause the stream to break through into the depressed area and take a new route to its final destination. All alluvial valleys are replete with evidences of old stream beds where this has occurred.

In a case of this kind, man can cooperate with Nature by facilitating the passage of the excess waters into the depressed areas, taking proper measures for their return to the original stream when the emergency has passed. This has been done in various places, notably on the lower Mississippi and Sacramento rivers. Where the land is of value for agriculture, as in the Sacramento Valley, a definite channel for the excess waters is segregated by levees, thus providing full reclamation for the major portion of the basin lands and permitting restricted use of the lands within the by-pass. When floods of great intensity are of such infrequent expectation that the partial reclamation of a portion of the by-pass lands is economically feasible, that channel may be divided, a fuse-plug levee permitting full flooding only in extreme cases. Such a scheme of control was recommended by the writer for the San Joaquin Valley lands, where winter floods of maximum intensity and short duration were of

low frequency, whereas the summer floods of fairly uniform intensity and longer duration were quite frequent.

In connection with by-pass channels, particular emphasis is laid on the fundamental principle of stability. Instances have come to the writer's notice where relief channels have been located on high land, either on account of lower right-of-way values or for some geographical or cultural reason. Unless one is willing to undergo the hazard and expense of fighting against natural laws, the location should be made with due regard to the course which the water would take if left to itself, also giving consideration to the feature of uniformity of gradient.

The writer considers that flood protection is one of the most important and most interesting studies now available to the engineer, and the great source of interest is the fact that each instance presents its individual problems to which no standard rules or formulas may be applied.

WILLIAM P. CREAGER,[13] M. AM. SOC. C. E. (by letter).[13a]—This excellent and comprehensive Report leaves very little to be added. However, there are one or two items which can be expanded.

In paragraph 24 of the Report, the Committee differentiates between "storage reservoirs" and "retarding basins" as applied to flood-control reservoirs. In paragraph 32 it is stated that the distinguishing feature of the retarding basin is its uncontrolled outlets. Therefore, it is assumed that storage reservoirs have controlled outlets.

The writer can see no logic in standardizing these two terms. To remove the control gates from an existing flood-control dam would not change the name from "storage reservoir" to "retarding basin." In many cases, both controlled and uncontrolled outlets are being provided for the same dam. The choice of a name for such a reservoir or basin would then be difficult.

It is believed that the advantages of controlled outlets have not been sufficiently emphasized, since they have two very important advantages which, for all except the smallest reservoirs, far overshadow their only disadvantage of greater cost of installation and operation. In the case of controlled outlets, the gates are kept wide open until the discharge reaches channel capacity. They are then gradually closed as the water rises in the reservoir in order to maintain an outflow which is the maximum possible without damage.

If uncontrolled outlets are limited in size to maintain a safe discharge when the reservoir is full, or nearly full, their capacity is entirely too small when the reservoir is starting to fill. Thus, when the reservoir starts to fill, a part of the discharge which controlled outlets could pass safely is held back by the uncontrolled outlets and reduces the effective capacity of the reservoir.

It is frequently the case, both when the reservoir is being filled and when it is being emptied, that delayed floods from tributaries below the reservoir require a considerable reduction in discharge at the reservoir, if not complete closure of the gates, until the peak of lower tributary floods has passed down-

[13] Cons. Engr., Buffalo, N. Y.

[13a] Received by the Secretary April 26, 1940.

stream points. This method of operation is impossible with uncontrolled outlets.

Another point which might be expanded is that conservation storage in a multiple-purpose reservoir in some cases can be allowed to encroach somewhat on the storage space allocated to flood control during certain seasons of the year. This method of operation is permissible on the larger streams where there is very definitely a season during which large floods occur and another season during which floods are much reduced. In the latter season the entire space allocated to flood control is not needed, and a part of it can be used to catch the smaller freshets to be released during the dry season.

This method of operation is being used at the Norris Dam of the Tennessee Valley Authority and the Tygart Dam near Pittsburgh, Pa.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FOUNDATION EXPERIENCES, TENNESSEE VALLEY AUTHORITY

A SYMPOSIUM

Discussion

BY MESSRS. BERLEN C. MONEYMAKER, R. F. WALTER, WILLIAM F. PROUTY, JACOB FELD, AND A. WARREN SIMONDS

BERLEN C. MONEYMAKER,[22] ASSOC. M. AM. SOC. C. E. (by letter).[22a]—The site of Chickamauga Dam on the Tennessee River just upstream from Chattanooga is of particular interest to both the engineer and the geologist because of the difficult foundation problems it presented. The choice of the dam site was limited to a relatively short section of the river which did not afford a wholly satisfactory location. Two sites had been drilled by the Corps of Engineers, U. S. Army, before the Tennessee Valley Authority (TVA) was created, and four additional sites were subsequently explored by the Authority. Although the site finally chosen was obviously the best one available, certain defects of a geologic character were recognized from the outset.

As Mr. Fox has pointed out, Chickamauga Dam is located on the great variety of thin-bedded limestones with thin, interstratified layers of bentonite and shale which are known collectively as the Chickamauga limestone. The Chickamauga limestone is in fault contact with the Knox dolomite about 3,250 ft upstream from the dam. This major thrust fault represents a stratigraphic displacement of about 5,000 ft, and as the Knox dolomite was thrust over the Chickamauga limestone, both formations are deformed for a considerable distance on either side of the fault. Many very sharp folds and even a greater number of faults occur in the rocks on which the various parts of the dam and lock are located. There were no outcrops of bedrock except along North Chickamauga Creek, at the extreme north end of the dam, where several very sharp, asymmetrical anticlines and synclines were exposed. In the river channel and flood plains, rock was covered by alluvium (clay, silt, sand, and gravel), which was more than 50 ft thick in some places. There was a thin, discon-

NOTE.—This Symposium was published in March, 1940, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: May, 1940, by Messrs. George K. Leonard, and F. B. Marsh.

[22] Senior Geologist and Asst. Chf. Geologist, TVA, Knoxville, Tenn.

[22a] Received by the Secretary March 20, 1940.

tinuous cover of residual clay at the extreme north end of the dam, and both residual materials (mostly clay) and terrace gravel overlie bedrock at the extreme south end of the structure.

Because of the very complex geologic structure and the great thickness of of the overburden, the geologic conditions were very difficult to determine. Even after several thousand feet of drilling was done, little was known definitely except that the upper portion of bedrock was generally quite cavernous. At this stage, Mr. Fox was assigned to the Chickamauga project as the resident geologist. By very careful and painstaking study of the cores recovered from the drill holes, he was able to discover a very thin layer of shale and bentonite sufficiently different from all other rocks to be recognized wherever found. The value of this layer as a "key" bed was immediately appreciated. After this discovery was made, the cores recovered from all previously drilled holes were restudied in detail, and within a very short time all but the more minute details of geologic structure were developed. At the same time that the key bed was discovered, it was also discovered that, although the rock above it was everywhere quite cavernous, the rock below it was entirely free from cavities except along a fault or two. This made possible the very accurate prediction of the elevation, at any point in the dam site, below which the rock was free from cavities.

The cavities constituted the most perplexing and the most serious of the foundation difficulties. They were developed mainly along joints and faults, although some of the larger ones on the south (left) side of the river exhibited a marked tendency to follow certain of the more soluble beds. On the whole, the cavities were very irregular in form. Some of them were continuous for appreciable distances both laterally and vertically. A cavity at station 55/24 extended downward to El. 510, a few feet below the level of the Wheeler Reservoir. Hundreds of small cavities were encountered throughout the dam site at various elevations both above and below the water table. The incorrectness of the concept long held by geologists that cavities may not be formed below the water table is nowhere better demonstrated than at Chickamauga. At this locality, the depth to which cavities were formed was determined, not by the position of the water table, but by the position of a layer of bentonite and shale, designated as the key bed, which effectively shielded the pure, underlying limestone from solution.

The early recognition of the seriousness of the foundation conditions at Chickamauga Dam was very fortunate. By taking great care in the exploration, preparation, and treatment of the foundation, a safe dam has been constructed on a very unsatisfactory site.

In his excellent paper, Mr. Lewis presents an interesting and useful record of the foundation treatment and rim tightening work done by the TVA at Norris Dam. Although the paper treats all phases of the work, this discussion is restricted to those bearing upon foundation conditions.

As might have been pointed out by Mr. Lewis, the site of Norris Dam was selected by the U. S. Corps of Engineers previous to the creation of the TVA. A considerable amount of preliminary investigation and exploration, on the basis of which the feasibility of the site was determined, was done by the

Corps of Engineers. Further exploration by the TVA did not materially alter the picture of foundation conditions afforded by the previous work.

The entire dam site is in approximately the middle third portion of the Copper Ridge (lower Knox dolomite) which was designated as the Rockhouse member by Arthur Keith.[23] The formation is about 1,000 ft thick at Norris Dam, and consists mainly of cherty dolomite and cherty magnesian limestone. As a whole, the formation is quite massive in character, the individual strata ranging in thickness from a few inches to about 20 ft. The strata dip rather uniformly downstream (southeast) at an average inclination of 4°, the strike being N. 65° E.—approximately parallel to the axis of the dam. Stratification planes were well marked throughout the formation. In the upper portion of bedrock, contiguous strata were usually found to be separated by thin layers or seams of residual clay. At greater depths, however, the strata were usually in close contact, although separated by a distinct break. In nearly all cases, these planes were sufficiently open to be water bearing, or potentially water bearing.

The rock involved in the dam site was entirely free from faults, although it was extensively jointed. Most of the joints were merely incipient structures, along which the rock broke on blasting, but some of them were fissures, commonly somewhat enlarged by solution. Some of the joints thus enlarged were open, but in most cases they were more or less completely filled with residual clay. Joints exhibited a great diversity of trends, but most of them were characterized by northeast trends and steep dips.

As is almost always the case in limestone and dolomite, the rock forming the foundation, abutments, and adjacent portions of the reservoir rim was, to some degree, cavernous. This condition was clearly recognized from the start, and construction was not undertaken until its seriousness had been fully ascertained. With the exception of a cave—6 to 10 ft wide and 10 to 25 ft high—in the right (west) abutment at El. 980, the openings resulting from solution of the rock were found to be of relatively small dimensions, except for lateral extent. Cavities were appreciably larger and somewhat more abundant in the abutments than in the foundation. Drilling revealed that cavities and seams ranging in vertical dimensions from less than 0.1 to 3.0 ft occurred in the foundation to a maximum depth of 86 ft below the original bed of the river. These cavities averaged slightly more than 0.3 ft in vertical thickness, and most of them were filled or partly filled with residual clay. Nearly all of the small cavities encountered in the foundation seemed to represent merely enlarged stratification planes, but the larger ones in the abutments were, for the most part, developed along joints. Some strata were found to be decidedly more cavernous than others because of their greater susceptibility to solution.

The cavities were not sufficiently large, open, or closely spaced in either the abutments or foundation to render the rock incompetent to bear the load to be imparted by the structure. Nevertheless, they constituted serious leakage hazards and made necessary the extensive and systematic treatment

[23] "Geology of Norris Dam Site," by Arthur Keith. Unpublished report to TVA, August 30, 1933, p. 7.

with which Mr. Lewis' paper deals. Without such remedial treatment, leakage undoubtedly would have occurred as soon as the reservoir began to fill. As leakage progressed, the clay filling would have been washed out of the cavities and seams, resulting in the loss of increasing volumes of water. The cost and difficulty of stopping, or even appreciably retarding, leakage that has developed in limestones or dolomites under and around a large dam can be thoroughly appreciated only by one who has attempted such a task.

The systematic washing of clay out of seams—most of which were along bedding planes and consequently inclined downstream at an average of 4°—and refilling the openings with grout removed the very slight possibility of failure by sliding within the foundation and the probability of settlement under the load of the dam, as well as leakage hazards.

The advantageous use of large-diameter drill holes in foundation exploration and foundation treatment is justifiably emphasized by Mr. Lewis. Norris is one of the first large dams in the United States at which large-diameter core drilling was done on a moderately large scale for both exploration and foundation treatment.

The paper by Mr. Ross also deals with a dam site located on limestone. Although the Guntersville site is quite different from both the Chickamauga and Norris sites, it has some geologic features in common with each of them. On the one hand, it resembles Norris in the simplicity of the geologic structure and in the relatively small amount of deep cavitation. On the other hand, it resembles Chickamauga in that much of the limestone is quite pure—and consequently quite soluble—and in the presence of a shaly stratum which has effectively controlled the depth to which solution cavities were formed.

During the early stages of the preliminary studies, the Bangor limestone as a foundation rock was looked upon with considerable suspicion. This suspicion was not entirely unfounded, for the largest caves in the entire TVA area are in the Bangor limestone. Moreover, the cavernous character of the rock is well attested by the enormous leakage under the only dam in the area to be founded upon the formation. It was not until seven other sites had been found unsuitable that the Guntersville site, previously known as the Coles Bend Bar site, was tentatively selected. A considerable amount of core drilling revealed that the site as a whole was unusually good.

As Mr. Ross clearly points out, the only serious foundation problems were those involving solution cavities, nearly all of which were above the shaly stratum. The most serious problem was occasioned by deep and extensive zones of solution under the south (left) earth embankment. Remedial treatment of the south earth embankment area was accomplished mainly by excavation to good rock and backfilling with impervious material and by grouting. The general condition of the upper portion of bedrock—which prevailed over much of the dam site—is well illustrated in a block diagram (Fig. 27) and a photograph (Fig. 28).

R. F. WALTER,[24] M. AM. SOC. C. E. (by letter).[24a]—The interesting paper by Mr. Lewis comes at a time when foundation treatment is steadily becoming a more important factor of dam construction due to the decreasing availability of ideal sites. The constant desire for additional safety makes careful foundation exploration and thorough treatment a prerequisite of modern dam building. The author's account of the work at Norris Dam, together with his description of methods used in obtaining the information necessary for adequate treatment, makes this paper of value to every engineer connected with foundation work.

No two dam foundations are alike but many may have characteristics similar to others. Solution channels through limestone were also characteristic of the formation underlying the immediate foundation of Alcova Dam in Wyoming. This dam, an earth-fill structure with a maximum height of 265 ft, a length of 763 ft, and a total volume of 1,600,000 cu yd, was placed on a foundation consisting of two formations dipping about 15° downstream. Thin-bedded, limey sandstone composes the abutments and immediate foundation for the dam, but this in turn is underlaid by similarly tilted thin-bedded limestone containing numerous solution channels. With the exception of one cavern under the upstream toe of the dam, which was found to contain fine sand, the solution channels were entirely clean, most of them containing warm water under a pressure of about 20 lb per sq in. as measured at original river level, and therefore required no washing as did those at Norris Dam. Warm water at the rate of approximately 10 cu ft per sec was discharging into the foundation area.

The problems at Alcova were (1) to intercept and stop the inflow of water into the foundation area under the impervious section of the dam; (2) to consolidate this same foundation area; and (3) to provide impermeable cutoffs under the dam to prevent loss of reservoir water. All of these problems were solved successfully by pressure grouting. Only the grouting operations required to stop the inflow of water into the foundation area will be discussed.

Similarly to the reservoir rim tightening at Norris Dam, it became evident almost immediately that considerable quantities of grout would be required to fill the numerous solution channels, and it seemed advisable to use a grout containing sand. Fortunately, there were deposits of a fine dune or blow sand just downstream from the dam site which only required screening through a common window screen (to remove an occasional rock) to obtain a sand that, when added to the grout, could be pumped through the regular piston-type pumps commonly used in pressure-grouting operations. Thereafter, as soon as it was established that a hole was connected to a solution channel, a grout consisting of 1 part cement to 2 parts fine sand and 2 to 3 parts water (by volume) was used. It is to be noted that a grout containing sand was used only in those holes that took grout freely and, in nearly all cases, in considerable quantities. An attempt was made to use the sand mixture in grouting the

[24] Chf. Engr., Bureau of Reclamation, Dept. of the Interior, Denver, Colo.

[24a] Received by the Secretary April 22, 1940.

seams and joints in the sandstone but this was quickly abandoned as it was found that there was a tendency for the holes to plug before the seams were thoroughly filled.

The nature of the formation made it imperative that all the grout possible be injected into each hole as a parallel hole 5 ft distant would often miss the irregular pattern of solution channels. Thus there was no way of beginning to grout at each end of a solution channel and closing in toward the center as was done on the curtain grouting at Norris Dam. Individual holes were stage-drilled and grouted to completion before another hole within 40 ft was allowed to be drilled, except in cases where there were two or more grouted holes intervening. The major volume of grout placed in the solution channels was injected through 10 holes which took a total of nearly 140,000 cu ft of the sand mixture as measured prior to mixing. The largest quantity injected into one hole was 38,021 cu ft. No concern was felt over the possibility of grout traveling into regions outside the dam area as it was realized that the only way of being positive that all openings were completely filled was to continue injection to refusal. Furthermore, any grouting unavoidably accomplished outside of the immediate foundation would tend to augment the seal and, therefore, would be not entirely without value.

As grouting progressed, cores were recovered containing grout varying from a thin film to a maximum thickness of 27 in. These cores indicated that a good dense grout had been deposited and showed only very slight evidence of the fine sand having segregated from the mixture. In cases of large openings the first grout placed was found to have shrunk somewhat, leaving a small passage between the top of the grout and the overlying rock. These passages were grouted with a neat cement grout using a water-cement ratio (by volume) of from 3 : 1 to 1 : 1. As the work neared completion, many grout cores were recovered having good bond with the overlying as well as with the underlying rock. The best criterion for ascertaining the effectiveness of the grouting was the reduction of water flowing into the excavated foundation area. This was finally reduced from an estimated flow of 10 cu ft per sec to less than 15 gal per min.

William F. Prouty,[25] Esq. (by letter).[25a]—The paper by Mr. Fox concerning the foundation of Chickamauga Dam, in Tennessee, demonstrates the benefit of close harmony by the engineers and the geologists, in dam construction, and the great importance of the accurate determination of geological formations and structure before foundation excavation is begun. It also demonstrates the great importance of continued study by the geologist during the entire period of construction. The present is an era in which engineering projects involve such great expenditures and such far reaching public liabilities that it becomes necessary to have every possible check on both permanence and safety.

At Chickamauga the close relation of major unsoundness to structure and formational character is revealed strikingly. The excellent cross sections,

[25] Head, Dept. of Geology, Univ. of North Carolina, Chapel Hill, N. C.

[25a] Received by the Secretary May 3, 1940.

showing in great detail the thickness, character, and structural weakness lines in the foundation, made possible accurate planning in regard to methods and costs of excavation and foundation improvement. Both the engineers and the geologists are to be congratulated on solving the foundation problems at the Chickamauga dam site.

The detailed geological studies at this locality, based on many deep borings, have brought out or emphasized a number of interesting facts: The bentonite beds are thicker, more numerous, and occur through a wider geological range than was previously thought; the bentonite beds also have great variability in regard to physical properties and permeability.

The so-called "key shale" or bentonite beds at the top of the lower Trenton limestone formation has limited the downward solution in the upper Trenton limestone throughout the area of its occurrence. The deep solution areas are associated with, and chiefly overlie, the zones of shear or intense strain. These lines of unsoundness in the Chickamauga area result from intense northwest-southeast compressional forces. The major overthrust from the southeast has its outcrop a short distance to the southeast of the south abutment of the dam. The numerous secondary faults of comparatively small heave and throw are largely parallel to the major fault and, like it, tend to have less angle of inclination in the near surface zone. It is interesting to note also that in the lower Trenton and upper Lowville, where the bentonite beds are developed, numerous minor faults are to be found frequently with steeper angles of inclination than the main faults and extend only between the beds. This has been caused, apparently, by the greater ease of thrusting along the weak bentonite beds. This type of faulting is particularly well developed where the more resistant member lying between the bentonite beds thickens by lensing in the direction of the thrust. Another type of faulting illustrated in the cross sections is that caused by local unsymmetrical folding, resulting in a greater thrust of the overlying beds in front of (northwest of) the fold.

The deep solution in the limestone in the area of Chickamauga Dam, as revealed by the core drill, bears out observations made in other limestone areas that solution is not limited, as thought by many, largely to the level of the present water table. There must be some deep-seated artesian flow in the area, or the area was once more highly elevated and the water table relatively lower than now. Although the latter explanation is a possible one, it is less probable than the first explanation.

The geological records from such extensive drillings are of value not only to the engineer in connection with the construction of the Chickamauga Dam, but the generalizations to be drawn from the numerous detailed cross sections produced are of such value as to be of use to the engineer in other areas with somewhat similar rock and structural conditions.

JACOB FELD,[26] M. AM. SOC. C. E. (by letter).[26a]—The Symposium papers give a complete description of the investigation, plan of treatment, and cure of the rock rebuilding under the three dams in the Tennessee Valley. The reports

[26] Cons. Engr., New York, N. Y.

[26a] Received by the Secretary May 17, 1940.

will become the standard reference for any future problems of similar nature; the great detail in which the various schemes are given will eliminate many costly researches and experimental developments for such future problems. The idea of strengthening rock for foundation purposes, as well as for the reduction of porosity, on such large scales, is revolutionary. In previous years, the discovery of such porous rock conditions would have meant the abandonment of the site.

It would aid the reader in establishing his perspective if the total cost of exploratory work and grouting were given for each dam, together with the corresponding total cost.

The solution channels were apparently sub-bed flow channels. What changes in the ground-water levels and in the ground itself are expected downstream of the dams, where a complete stoppage of such subterranean flow has been provided?

Mr. Ross' discussion of the economy of inert admixtures to the grout (see heading, "Use of Rock Flour"), pointing out that such admixtures retard the time of set and therefore that greater areas are treated from any one hole, might be used as an argument for the use of such admixtures with an increase in hole spacing. If the admixtures are finer than cement, especially if they are of colloidal clay (such as bentonite), the grout will travel through finer crevices and will seal such crevices not only by the setting of the cement but also by the swelling of the colloidal gel. Such admixtures also eliminate the formation of fine shrinkage seepage channels in the cement grout, through which water flow will continue and tend to dissolve the limestone, as well as the cement. Water in fine channels gives greater surface contact with the limestone per unit volume and thereby speeds the rate of chemical solution.

The depletion of ground water from the subsurface below each dam during low-water periods will be followed by rapid filling of the voids after heavy rains. The surfaces in the cavities have meanwhile been exposed to drying and oxidation. Has any provision been made for keeping the ground-water table sufficiently high below each dam to eliminate such alternating drying and wetting, which would accelerate future increase in ground cavities?

A. Warren Simonds,[27] Assoc. M. Am. Soc. C. E. (by letter).[27a]—A valuable and instructive treatise on the art of foundation grouting of dams is contained in the paper by Mr. Lewis. The proper foundation treatment is one of the major items of importance in the construction of modern dams. If this work is done properly at the time the dam is built, it will minimize the expense and effort of regrouting the foundation at some later date after the reservoir has filled. Moreover, regrouting the foundation beneath a dam under water load may cause undesirable alteration of the distribution of stress at the base of the dam as a result of changing the distribution of uplift pressure. Therefore, an effective job of foundation grouting at the time of construction is desirable.

The problems connected with foundation grouting vary widely; at one dam site they may be radically different from those at some other site. This

[27] Engr., U. S. Bureau of Reclamation, Denver, Colo.

[27a] Received by the Secretary May 29, 1940.

variation makes it necessary to alter the grouting procedure to fit the field conditions as the occasion demands. Occasionally it is necessary to vary the grouting procedure at adjacent holes in the same formation.

Mr. Lewis states that the foundation at Norris Dam was composed of a hard banded dolomite with numerous seams, some of which were partly filled with clay and some partly open. In grouting seams partly filled with clay, the seams should be well washed before injecting cement grout. If cement grout becomes mixed with clay, the resulting mass may be slow in setting and it is doubtful if such a mixture will develop much strength.

The patterns of wash and grout holes at Norris Dam thoroughly covered the foundation. The exploration of the foundation seams presented numerous problems which were ingeniously solved by the use of mechanical feelers or "hole explorers" and periscopes. The use of expanders or packers in washing the seams would insure the injection of wash water and air into the seam. In washing seams the question often arises as to the thoroughness of the washing operations. There is a possibility that the air and water may cut channels through the clay into the adjacent holes without completely washing the seam.

It is noted that upheaval gages were used to detect small movements in the upper bedrock so that careful control could be made of the grouting pressure. The ideal pressure used in injecting cement grout in foundation rock is the maximum that can be sustained without causing lifting or displacement of the foundation rock. This results in a wider dispersion of the grout thereby permitting a maximum benefit to be obtained from each hole drilled. Since there are no set rules for the establishment of a safe maximum pressure, the use of mechanical devices for the purpose of detecting small movements of the foundation is beneficial in regulating the grouting pressure. Surveys for the control of uplift movement have been made with precise levels, Whittimore type strain gages, extensometers, tiltmeters and, if the dam is under load, with uplift pressure pipes. The extent of the observations for uplift depends on the type of rock in the foundation and on the external load applied.

The grouting equipment described by Mr. Lewis is of the general type now in use at most grouting jobs. The air-driven or steam-driven duplex reciprocating pump is the most satisfactory machine for placing cement grout. The 7 by 5 by 10-in. size used at Norris Dam is not as flexible as the 10 by 3 by 10-in. size and may not be as satisfactory except in grouting large cavities at low pressure. In place of cast-iron fluid pistons with rubber packing, the writer believes that a medium-hard rubber piston on a steel core is more satisfactory. These pistons are slightly flared at the ends so as to produce a wiping action in the cylinder. The liquid end cylinders of the grout pump should be equipped with liners having hardened steel sleeves. Mr. Lewis states ("Shallow Grouting: Equipment") that "Piston packing and valves were usually replaced in the field, although it was necessary to have cylinder liners removed and replaced in the machine shop." Pumps are now being constructed in which the liners can be removed and replaced in the field, thereby eliminating the necessity of machine shop service. The writer agrees

with Mr. Lewis in his statement, "Valve disks of medium rubber were found to be far superior to the fiber disks furnished as original equipment." A more satisfactory arrangement of the equipment used at Norris Dam would have included a mechanically agitated sump between the mixer and the agitator as Mr. Lewis suggests.

The water-cement ratio of grout used in foundation grouting is dependent on the type of rock, presence of seams, cavities, zones of broken rock, etc. The purpose and location of the grout holes are also factors in determining the most suitable grout mixture. The most successful grout mixture is the thickest that can be injected without plugging the hole.

An excellent description of the use of packers in grouting operations is given by Mr. Lewis. There is considerable variance of opinion among engineers as to the relative merits of packer grouting and stage grouting. The procedure in grouting the foundation at Grand Coulee Dam combines the experience of the U. S. Bureau of Reclamation in grouting at other Bureau dams. Grout holes are drilled into the granite foundation until an open seam is encountered. This is known by the loss of the drill water or by a flow of seepage water into the hole. The hole is then grouted progressively in stages; that is, the drilled part of the hole is grouted, the grout is cleaned out of the hole after the cement has taken its initial set, and the next stage is drilling and grouting. A fish-tail bit is used in cleaning out the hole. If the drilled hole is tight, stage grouting with a packer is resorted to and the hole is grouted starting at the lower part of the hole and working progressively upward. Where the foundation rock is fairly tight packer grouting is probably more effective than stage grouting. Where several holes are interconnected by a common seam the use of packers may not be as effective as grouting in stages. Moreover, if a packer is placed in broken or caving rock there is a possibility of grout leaking by the packer and the packer becoming grouted in the hole.

The problems arising in foundation grouting vary widely. At times the engineer in charge may find his ingenuity taxed to the limit in order to effect a successful job of grouting. Mr. Lewis shows that he has given considerable thought and study to the problems arising in treating the uncertain character of the foundation of Norris Dam. The results obtained are worthy of study by all engineers engaged in foundation work.

The paper by Mr. Hays adds needed information on the treatment of the foundations of dams as such material now available to the engineering profession is quite limited. It contains an excellent description of the grouting of a difficult foundation for two types of structures: Concrete structures, which include the lock, spillway, and power house; and the earth structures which include the two embankment dams at each abutment. The paper also includes a description of some experimental grouting under the embankment sections and a description of the temporary construction grouting.

It is interesting to note that the purpose of the experimental grouting under the embankment sections was to determine the most suitable method for making the cutoff under the earth dams. Grouting a foundation which contains numerous small defects under a cover of 40 ft of overburden presents

numerous problems and questions to the engineer. One of the most important questions relates to the problem of under-grouting or over-grouting. If the injection of grout is terminated too soon, an ineffective job of grouting will result. If the grouting is excessive, cement will be wasted and the costs of grouting will be increased unnecessarily. Successful grouting of test areas, as was done at Chickamauga Dam, gives confidence to the engineer in charge of the work.

The regular program of permanent grouting under the concrete structures, as described by Mr. Hays, is of the type now generally used in the construction of most dams built by the U. S. Bureau of Reclamation. First, a general consolidation of the surface rock is obtained by blanket grouting through shallow holes. This is followed by grouting through holes of intermediate depth, and finally the deep holes forming the cutoff are grouted. An interesting feature described by Mr. Hays is the inclination of two sets of deep holes in opposite directions in the plane of the cutoff. This procedure is effective in intercepting vertical as well as dipping seams.

The grouting at Chickamauga Dam shows the results of careful planning and execution. The thoroughness of the work is indicated by the re-drilling and grouting of the consolidation system of holes as many as five times in certain areas. The final spacing of drilled, washed, and grouted holes of 12 in. on centers should have reduced the possibility of missing small cavities and solution channels to a minimum.

The program of grouting under the earth-dam sections at the abutments presented numerous complications due to seepage water and a badly distorted geological structure. It is interesting to note that asphalt grouting was tried (and was not successful) in an attempt to grout an underground channel beneath the south earth dam. The development of low-cost grout composed of a mix containing 4 parts sand, 1 part bentonite, and 2 to 3 parts of cement is similar to the mix used by the contractor in grouting the leak in the cofferdam at Grand Coulee Dam where sawdust and shavings were also added.

The use of bentonite combined with cement as an ingredient of a grout mix has been used successfully in temporarily shutting off flows of water. However, it is questionable whether a bentonite-cement combination makes a permanently stable mix. The same question may arise in the case of the clay-cement combination when subjected to repeated soaking and drying.

It is noted in the discussion of the temporary construction grouting of the first stage cofferdam that 36,350 cu ft of asphalt and pitch were used. It is regretted that Mr. Hays did not describe the method used in injecting this material and the results obtained.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

GENERAL WEDGE THEORY OF EARTH PRESSURE

Discussion

By Donald M. Burmister, Assoc. M. Am. Soc. C. E.

Donald M. Burmister,[25] Assoc. M. Am. Soc. C. E. (by letter).[25a]—In his presentation of the general wedge theory Professor Terzaghi has made a most important contribution to current knowledge of earth pressure and has furnished a more rational basis for the design of retaining structures. The important question now is how can this information be applied practically and in what way should present methods of design be revised? It is hoped that the author will formulate a few principles and criteria for the design of retaining walls, abutments, and similar structures as a guide for design in the future. In the analysis of a given situation a number of questions arise:

(1) What reasonable assumption can be made regarding the angle of internal friction and of wall friction to be used in the computation of earth pressure?

(2) What design criteria and factors of safety should be used to insure permanent stability of a retaining wall?

(3) How do conditions change with time for the different classifications of soils and what minimum requirements for drainage should be specified in the different classifications of soils?

(*1*) *What Reasonable Assumption Can Be Made Regarding the Angle of Internal Friction and of Wall Friction to Be Used in the Computation of Earth Pressure?*— The values depend not only on the character of the backfill material and the moisture content, but also on the method of placing and amount of compaction, and whether the fill has been built up toward or away from the wall, or has been built up in level layers. It may be doubtful whether these conditions can be known in sufficient detail in a given case to make a reasonable estimate of the possible limits within which the angles of internal friction and of wall friction may lie. If average values must be used, in the absence of specific information, a tabulation of the possible range of values for the different classifications of soils and for the different methods of backfilling would be very useful, par-

Note.—This paper by Karl Terzaghi, M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Howard F. Peckworth, M. Am. Soc. C. E.; February, 1940, by Jacob Feld, M. Am. Soc. C. E.; and April, 1940, by M. G. Spangler, Assoc. M. Am. Soc. C. E.

[25] Asst. Prof., Civ. Eng., Eng. School, Columbia Univ., New York, N. Y.

[25a] Received by the Secretary May 2, 1940.

ticularly the possible lower limits that should be assumed. If tests are to be made to determine the values, it would be desirable to know what type of test would be considered satisfactory to determine the limits, and how many tests should be made in order to obtain representative values.

(2) *What Design Criteria and Factors of Safety Should Be Used to Insure Permanent Stability of a Retaining Wall?*—The investigation of stability involves a consideration of a number of important factors. It is obvious that the usual criteria for stability against overturning are inadequate. A consideration of stress-strain phenomena in soils shows that a retaining wall founded on ordinary soils will both settle and tilt a certain amount. The logical design criteria, in the light of this knowledge and of the important influence of wall yield, as demonstrated by Professor Terzaghi,[2,6] are to limit settlement and tilting to such values that the safety of the wall is assured under the most severe conditions to be anticipated, and also that the appearance of the wall is not damaged. The backfill is not an ideal, cohesionless, homogeneous material, the method of placing the fill may have an important influence, and the frictional resistance may not be fully mobilized. It seems logical, therefore, to assume that the center of pressure on the back of the wall should be taken at least at a height of 0.4 H, for the computation of stability, $\Sigma H = 0, \Sigma V = 0, \Sigma M = 0$, and that the base width should be increased so that the resultant pressure on the base computed for the ordinary Coulomb earth pressure would fall between the 0.55 and 0.60 point of the base width, instead of the usual two-thirds point, as shown in Fig. 11(*a*). This has two important effects: First, it makes the

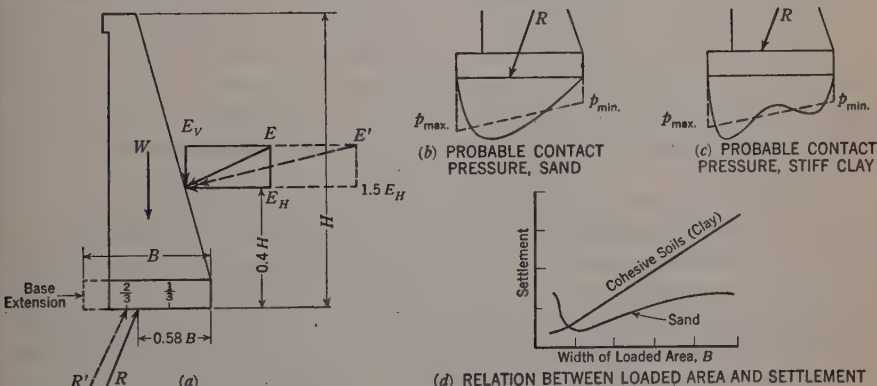


FIG. 11

settlement of the wall smaller and more uniform; and second, it reduces the maximum toe pressure and thereby reduces tilting. The pressure computed from Coulomb's general equation represents the minimum or ordinary lateral earth pressure condition, which obtains after the wall yields a certain amount, as demonstrated by the author.[6] However, some estimate must be made for the greatest pressure that is likely to occur. This can only be learned, as pointed out by Professor Terzaghi, by accumulating accurate, complete information on retaining wall failures and by carefully analyzing these situations

[2] "A Fundamental Fallacy in Earth Pressure Computations," by Karl Terzaghi, *Journal*, Boston Soc. C. E., April, 1936.

[6] "Large Retaining Wall Tests. I. Pressure of Dry Sand," by Karl Terzaghi, *Engineering News-Record*, February 1, 1934.

for the possible maximum earth pressure. In the absence of this knowledge it seems reasonable to make the simple assumption that the horizontal component of the earth pressure may conceivably be, for example, $1\frac{1}{2}$ times that computed from Coulomb's formula. This, in effect, means that the frictional forces are not fully mobilized from some cause and that both the angles of internal friction and of wall friction are low. Under these extreme conditions the resultant must not pass outside of the middle third of the base.

The next factors to be considered, and about which little is really known, are the distribution and intensity of the base pressure for an eccentric loading. The conventional trapezoidal distribution is probably modified in a very important manner, as indicated in Figs. 11(*b*) and 11(*c*). The maximum intensity may possibly be 25% greater than that computed by the conventional method. For a more rational design the load-settlement relations should be determined for the foundation material by means of loading tests, particularly for large retaining walls. The question then arises in the case of a long, continuous footing whether or not allowance should be made for the effect of the width of the foundation on settlement as is done in the design of spread foundations (see Fig. 11(*d*)), particularly in view of the eccentric loading. If presumptive, allowable bearing values must be accepted, it would seem logical to limit the maximum toe pressure by the conventional method to 75% of the allowable. Settlement and tilting records, together with tests by pressure cells placed under the base and on the back of the wall, would yield very valuable information on all of the factors that influence the situation, and would help to explain how conditions change with time from the beginning of the backfilling operation.

The condition for stability against sliding for the horizontal component of the earth pressure, times $1\frac{1}{2}$, is usually satisfied if other conditions are met satisfactorily. The important question here is the lowest value the sliding friction can assume for the different classifications of soils when saturated, and also how much passive pressure can be counted on under the most unfavorable conditions.

(3) How Do Conditions Change with Time for the Different Classifications of Soils and What Minimum Requirements for Drainage Should Be Specified in the Different Classifications of Soils?—The normal shrinkage or the swelling with saturation of certain soils affects both the magnitude of the earth pressure and the position of the resultant—certain cyclic effects. An important question in railroad and highway embankments is whether the full live load should be considered or only some fraction—for example, 20% in the computation of the earth pressure for retaining walls and abutments. Probably a large proportion of retaining wall failures have occurred during or just after a heavy rainstorm. If proper provisions for drainage have not been installed, hydrostatic pressure on the back of the wall would exert pressures for which few walls are designed. Professor Terzaghi has emphasized the importance of proper drainage and methods to secure it.[26] It is common practice on railroads to backfill abutments for some distance with cinders for drainage purposes, but it is not always possible to use such materials because they are not readily available.

In conclusion the writer wishes to thank Professor Terzaghi for his excellent and timely paper.

[26] "Effect of the Type of Drainage of Retaining Walls on the Earth Pressure," by Karl Terzaghi, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Vol. 1, Paper J-3, Cambridge, Mass., 1936, p. 215.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FLASH-BOARD PINS

Discussion

BY CHILTON A. WRIGHT AND CLIFFORD A. BETTS, MEMBERS, AM. SOC. C. E.

CHILTON A. WRIGHT,[15] AND CLIFFORD A. BETTS,[16] MEMBERS, AM. SOC. C. E. (by letter).[16a]—The interest shown in this paper by the discussions, and by the interesting side-lights revealed, is appreciated.

The description, by Mr. Field, of his flash-board design for handling cross drainage in an irrigation canal was most interesting. Calling attention as it does to the practical applications of flash-boards, this discussion corroborates the experience of others who, since publication of this paper, have advised the writers of the dependable operation of installations based on the paper.

The formulas developed in the paper will not apply to the pipes set horizontally in the abutments.

The allusion to inadequate capacities of many existing spillways in the light of recent flood data is very important and is receiving consideration by engineers as shown by Mr. Creager in a lecture to the Metropolitan Section of the Society on March 20, 1940.

Hinged Flash-Boards.—The replacements mentioned by Mr. Creager under the heading "Flood of January 1, 1934," become unnecessary if hinged flash-boards are used. Since the publication of the paper (in May, 1939) a large number of such installations have been made in various parts of the United States. Instead of being satisfied with charging the loss of flash-boards that float away at high water to the added power head obtained from the use of these flash-boards, even for a period of a few days, some power companies are using hinges and are saving the boards as well as the power.

Other Tests.—The method of test described by Mr. Ryder is ingenious, but it had been tried by the writers and found wanting. When the use of pipes for

NOTE.—This paper by Chilton A. Wright and Clifford A. Betts, Members, Am. Soc. C. E., was published in May, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1939, by Messrs. William P. Creager, Lincoln W. Ryder, and E. T. Schuleen; December, 1939, by John E. Field, M. Am. Soc. C. E.; January, 1940, by Julian H. White, Assoc. M. Am. Soc. C. E.; and April, 1940, by Harry H. Hatch, M. Am. Soc. C. E.

[15] Prof., Hydr. and San. Eng., Polytechnic Inst. of Brooklyn, Brooklyn, N. Y.; formerly Hydr. Engr., National Bureau of Standards, Washington, D. C.

[16] Engr., U. S. Forest Service, Washington, D. C.

[16a] Received by the Secretary May 3, 1940.

pins was first considered by the U. S. Forest Service, two specimens each of galvanized-iron pipe, 3 in. and 4 in. in diameter, were tested in a 200 000-lb machine on a set-up similar to that described by Mr. Ryder. The 3-in. pipe was placed on rounded supports on a 14.4-ft span and loaded in the center by means of a 4-in. pipe 20 in. long, representing the socket support. The distance from the socket pipe to the support was made equal to the computed distance to the point of application of the water pressure (flash-board height 3 ft, overtopping head of water 4 ft). The machine loads were divided by 2 to obtain an average value for each pipe as a cantilever.

The maximum stresses determined by this test (58 700 lb per sq in. and 47 300 lb per sq in. for the 3-in. and 4-in. pipe, respectively) were considerably lower than the later mechanical test results shown in Table 2(*b*). The 3-in. pipe attained a maximum stress of 74 000 lb per sq in. in the mechanical test and 73 000 lb per sq in. in the hydraulic tests (Table 2(*a*)).

The maximum stresses were obtained at a measured deflection of 17° which was the same value as shown in Table 2(*b*) for the 3-in. pipe. However, the pipes were not broken, although they were bent to the limit of the machine, 45 degrees. It is believed that this method of testing cannot be depended upon to give stresses suitable for use in the design of flash-board pins. Actual hydraulic tests are much to be preferred.

The discussion by Mr. Schuleen was much appreciated by the writers, who had previously had access to the comprehensive test data of the Pennsylvania Water and Power Company. Although the methods of mechanical testing described by Mr. Schuleen differed from those of the writers, the principal results (maximum loads and deflections) are in general agreement.

The writers are in full agreement, that the characteristics of standard pipe make it structurally and economically adapted to use for flash-board support. Tests of steel pins made prior to the pipe tests disclosed how a small variation in the chemical make-up of steel pins can have a large and hazardous influence on failure head in the field. Commercial pipe, on the other hand, has a tolerance that requires little checking.

The characteristic definite deflection for maximum moment is shown for the results of the standard pipe tests. The value of 20° agrees closely with the writers' values in Table 2.

Although Equations (19*b*) and (20) presuppose the direct use of the resisting moment, it is thought that the computation of the stress from the maximum moment (using a section modulus based upon the original dimensions of the pipe) is of value in two ways. It determines the variation among individual pipes of a given size and also aids in determining a suitable average value to be used in design with tabular standard values of section modulus for a specified pipe size. The cantilever loading method of making mechanical tests seems to agree better with the hydraulic tests than the double cantilever method of testing.

The original data loaned to the authors from the Pennsylvania Water and Power Company included a large number of actual field tests of pipe-supported flash-boards on the Holtwood Dam. The recorded failures were at heads giving

stress values in the pipes slightly lower than indicated in the mechanical tests of the company.

Mr. Hatch has reported an interesting field test on the Cobble Mountain spillway in which wrought-iron pipes were used. For purposes of comparison with the value of 62.9 kips per sq in. found by Mr. Hatch, the maximum stress computed by the simplified moment formula for Test 24 in Table 2 should be 56.3 kips per sq in. The value for Test 10*b*, 60.6 kips per sq in., was determined directly by mechanical test. Since steel pipe was much more readily available than genuine wrought iron, particularly in small sizes, it was adopted for test and was used by the Forest Service.

Pressure Reduction on Front of Flash-Boards.—In the data on actual dynamic pressure of the flowing water on the inclined flash-board (Fig. 15) it is to be noted that no noticeable variation from the assumed (static) pressure occurred up to a deflection angle of 20°. An interesting comparison is brought out by relating the maximum deflection attained before an appreciable departure from the theoretical value occurred, to the percentage of overflow head, $\frac{H - h}{h}$. The results in Table 9 show that, for all the overflow tests except possibly that of the 2-in. pipe, the error due to neglecting the dynamic effect of the water was negligible.

Mr. White's experiments were very interesting, showing an intelligent attack upon the perplexing question of the effect of pressure changes on flash-board action. Fig. 16, plotted in dimensionless terms, certainly emphasizes the hydraulic similarity of pressures on flash-boards made to different scales. For all ratios of length of boards to head on crest, the points seem to lie on the same curve (see Fig. 16). For any vertical board a pressure reduction of 10% can be expected at a distance below the top equal to 15% of the total head. This could be interpreted to mean that the pressure at a point 0.06 ft below the top of a 5-ft board with 0.4 ft of water going over the top would be reduced in the same amount as for the same point on a board 1 ft high with the same head. Although this seems difficult to believe, further inspection of Mr. White's tests might ascribe it to the added head of velocity of approach. The computation of the resulting total pressure would have to be made by integrating along the established pressure curve, such as, for example, Mr. White's determination of moment for Test 32, Table 2(*a*).

TABLE 9.—LIMITING DEFLECTIONS FOR STATIC PRESSURES

Pipe size, in inches	Schuleen		Wright and Betts	
	Overflow head, $\frac{H-h}{h}$, in percentages	Limiting deflection angle before departure of static pressure from theoretical value (in degrees)	Overflow head, in percentages	Deflection at failure, in degrees
2.5	....	....	8	17
1	22	34	22	33
1.5	....	....	35	22
....	39	26	....	....
....	56	18	....	....
2	....	....	66	21+
....	84	....	...	....

The reduction of pressure at the top of the flash-boards due to overflow, which is so well covered by Mr. White, is an interesting side-light but such a minor factor that it can be ignored except in unusual cases of very high approach velocities. For practical flash-board design the total head on the crest is sufficient.

Partial Vacuum Behind Flash-Boards.—Mr. Creager's able analysis of the lower stresses produced by flash-board pins tested under partial vacuum explains the variation of these from the writers' results. One of the reasons for running the tests described in the paper was the difficulty encountered when attempting to operate an installation based on Mr. Creager's table. The 42 000 lb per sq in. just did not apply to the standard pipe being used, which had a modulus of rupture of 70 000 lb per sq in.

The finding, indicated by Mr. Schuleen, that vacuum under the nappe decreased the moment approximately 7%, checks with findings at several installations that have been reported to the writers.

The paper did not attempt to analyze the effects of vacuum because in some cases on actual dams the pressure on the down-stream side of the boards is decreased, creating a "pull," whereas in others enough back-water is drawn up beneath the nappe to counteract this effect. Furthermore, the effect of vacuum can be readily corrected by the use of heavier pipe, the size of which can be computed from the observed failure head. Where it is impractical to ventilate the nappe, it is advisable to provide for larger sockets in case the heavier pipe becomes necessary.

As a practical matter, the degree of vacuum obtained would depend upon the height of the flash-boards, the clearance at end of the crest, the width of the flat channel immediately below the boards, and other factors, and should receive consideration in design of an installation. The effect of this vacuum would be to cause an earlier failure of the pins—that is, it would be on the safe side. Since an attempt to make an allowance for vacuum may very readily lead to the pipe failing at a higher head than desired, the recommendation in the last paragraph of Mr. Creager's discussion, which is also made in the original paper, should be emphasized.

In conclusion the writers wish to thank all those who have aided in shedding some light on the design of flash-board installations.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FIELD TESTS OF A SHALE FOUNDATION

Discussion

By Messrs. Jack R. Rountree, and August E. Niederhoff

Jack R. Rountree,[6] Jun. Am. Soc. C. E. (by letter).[6a]—The author is to be commended not only for presenting an interesting and valuable paper, but also for his purpose in placing before the profession a complete record of a carefully planned series of field tests. The results of only a few such tests have been published in engineering literature. In general, the physical properties of different shales are quite different; therefore it is seldom that the results of any one test may be correctly applied to a case at hand. A compilation of test results covering many types of shales, nevertheless, may be of considerable value in estimating the limits between which might fall the properties of a particular shale foundation.

This paper sets forth a very practicable field test for determining the resistance to sliding. The fissile character of some shales precludes the possibility of obtaining representative specimens large enough to be tested in the laboratory; in such instances, the engineer must necessarily resort to field tests in order to investigate the resistance to sliding. It is significant that the author recommends field tests to supplement laboratory tests, in "any comprehensive foundation investigation."

In many cases of dam construction, the contact surface between the foundation and the concrete is very irregular, and the problem of evaluating the resistance to sliding is reduced to a determination of the shearing strength of the foundation material. The sliding tests described by Mr. Niederhoff are well suited to the investigation of shearing strength of a shale foundation. At Test Block No. 8, where the underlying shale was described as "soft and inferior to that which ordinarily would be permitted under the dam," the failure evidently occurred within the shale, rather than at the contact surface; in this instance, the test results give a measure of the ultimate shearing strength of the "inferior" shale. In all cases where failure may occur within the shale it is believed advisable to remove lateral support by excavating a shallow trench

Note.—This paper by August E. Niederhoff, Assoc. M. Am. Soc. C. E., was published in September, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Harry H. Hatch, M. Am. Soc. C. E., and February, 1940, by Jacob Feld, M. Am. Soc. C. E.

[6] Assistant Structural Engr., TVA, Knoxville, Tenn.

[6a] Received by the Secretary May 6, 1940.

around the boundary of the shear area. For a large shear area, such as that beneath a monolith of a gravity type dam, lateral support around the boundary of the area may have only a negligible effect upon the total shearing resistance. For small areas, however, this effect may be significant.

A series of tests conducted at the site of Watts Bar Dam on the Tennessee River in Tennessee represents an example in which the test methods and apparatus described by the author have been adopted in the testing of a different type of shale foundation. Due to distinct differences between the Watts Bar and Possum Kingdom shales, the results of the two series of tests are not similar. The Watts Bar tests were conducted by the Tennessee Valley Authority, in conjunction with the U. S. Army Engineers, for the purpose of determining the shearing strength of the shale in a direction parallel to the bedding planes.

The shale underlying the easterly portion of Watts Bar Dam is a part of the Rome formation and is classified as a cemented, clay and silty shale. It is fairly compact and, in general, unweathered. The shale is characterized as "fissile" and is comprised of very thin layers which are easily separated; the surfaces of these layers are smooth and glossy in appearance. The shale is interbedded with thin layers of hard sandstone which vary from $\frac{1}{8}$ in. to 2 in. in thickness and which are normally from 1 in. to 24 in. apart. In general, the shale has a dip angle of from 20° to 30°, but occasionally, in limited areas, the bedding planes are horizontal. The apparent weakness of the material against movement parallel to the bedding prompted these tests.

Six concrete test blocks, 5 ft long, 3 ft wide, and about 4.5 ft high, were cast directly on the shale foundation. The contact surfaces were parallel to the bedding planes and were as nearly horizontal as the nature of the rock permitted. In contrast to the Possum Kingdom tests, where it was assumed that failure would occur by sliding of the concrete on the shale, the Watts Bar tests were

TABLE 4.—Test Data, Watts Bar Dam, Tennessee

No.	Description	Test Blocks Nos.:					
		1	2	3	4	5	6
1	Preliminary period of sustained vertical load, in minutes	65	70	55	74	None	None
2	Increments of horizontal load, in pounds	5 000	5 000	5 000	5 000	5 000	10 000
3	Average rate of application of horizontal load, in pounds per minute	9 700	7 300	1 600	1 700	6 500	6 600
	Intensity of Load, in Tons per Square Foot, Parallel to Contact Surface:						
4	At failure	3.08	5.06	1.84	3.03	1.83	2.20
5	Estimated failure under sustained load	2.46	4.13	1.51	2.46	1.50	1.81
6	First sliding	2.30	3.31	1.51	2.53	1.34	2.20
7	Second sliding	1.46	2.06	2.86	1.32	....	....
	Intensity of Load, in Tons per Square Foot, Normal to Contact Surface:						
8	At failure	5.71	4.36	2.07	5.10	2.03	4.88
9	Estimated failure under sustained load	5.65	4.26	2.05	5.06	2.02	4.86
10	First sliding	5.63	4.18	2.05	5.06	2.00	4.88
11	Second sliding	3.10	2.11	5.10	2.52	....	....

predicated upon the assumption that failure would occur within the shale at a short distance below the contact surface. Immediately prior to testing, the shale around each block was trenched out to a depth of about 2 in. below the projection of the contact surface. Trenches were located about 6 in. from the

edges of the blocks, and were carefully prepared by hand tools to insure that no shale under the contact surfaces was disturbed. At each block this trench served to remove lateral support from around a short prism of shale, thereby simulating the conditions existing for a large contact area, for which lateral support around the boundary of the area would afford very little, if any, additional shear resistance.

The methods of loading the blocks and of measuring the deformations were similar to those described by Mr. Niederhoff. The general procedure consisted of bringing the vertical load up to a specified intensity, and then loading the block with increments of horizontal load until failure occurred. Deformations were observed after each increment of horizontal load had been applied. The failure point was taken to be that point at which a constant horizontal load produced a continued movement of the test block. After failure, all loads were relieved; the vertical load was then brought up to a specified intensity, and the horizontal load applied gradually until a slow, uniform sliding of the block was produced.

The speed of testing used at each block varied somewhat, as shown by Items Nos. 1, 2, and 3 of Table 4. After testing, each block was tipped over to allow inspection of the failure surface. At all blocks, the failure surfaces were from $\frac{1}{4}$ in. to 2 in. below, and essentially parallel to the contact surfaces; the shale was well bonded to the under side of the concrete test blocks. Similar failure surfaces were observed under Blocks Nos. 1, 3, 4, and 5; at Block No. 2, the failure occurred along a decidedly irregular surface; at Block No. 6, the failure surface was more nearly plane than at any other block, due to the exceptionally smooth bedding at that location.

At Blocks Nos. 1, 2, 3, and 4, the vertical load was sustained for a short period before application of the horizontal load. The effects of vertical deformation during the short period of sustained vertical load probably increased somewhat the shear strength of the material; under a long-time loading, such as during the construction period of a large gravity-type structure, a similar but more pronounced increase might logically be expected.

Fig. 11 shows the average horizontal deformation plotted against horizontal jack load, for Test Block No. 4. Similar curves were observed at all blocks except at Block No. 6. Fig. 12 shows the variation in the instantaneous value of the rate of change of deformation with respect to the horizontal jack load for Test Block No. 4; this curve suggests that progressive permanent yielding (shearing off of projections on the failure surface) occurred only in the vicinity of the failure load. Construction of curves similar to Fig. 12 for other test blocks led to the adoption of a value of 83% of the ultimate horizontal load as the average point of "break" in the deformation curves; a sustained horizontal load of this amount might finally produce failure; therefore this reduced amount is a more conservative measure of the strength of the material than the ultimate failure strength indicated by the tests. Items Nos. 4 to 11 of Table 4 show the essential data obtained from the tests; the "estimated failure under sustained load" (Items Nos. 5 and 9) is based upon a horizontal load equal to 83% of the observed failure load.

In the interpretation of the test data, the internal friction theory as orig-

inated by Coulomb is used on the ground that it has been accepted by many engineers and that it affords a means of interpreting the data in a systematic manner. The resistance to movement is given by the expression:

$$s = s_1 + k\,q \qquad (5)$$

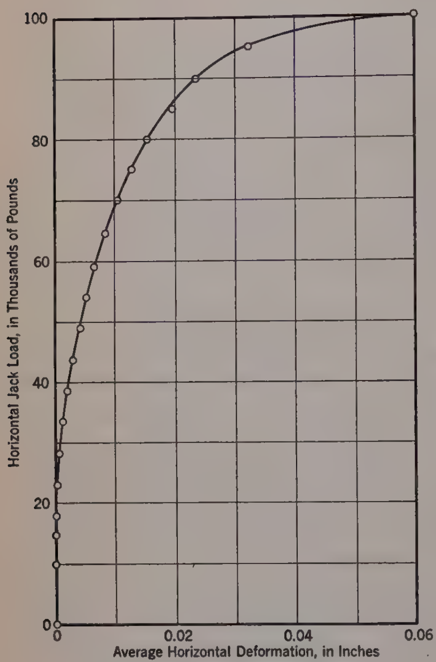


Fig. 11.—Average Horizontal Deformation, Test Block No. 4

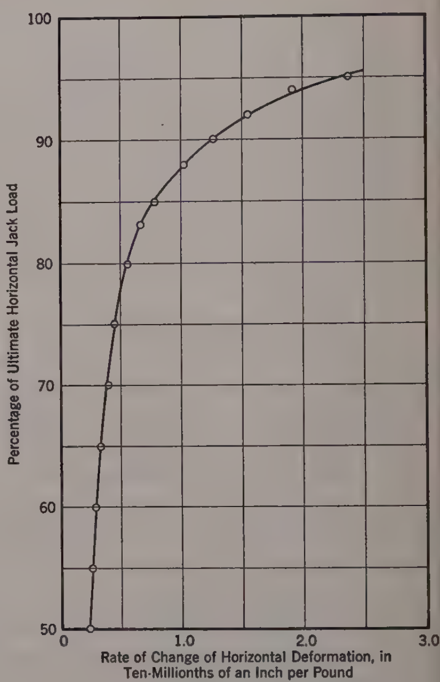


Fig. 12.—Rate of Change of Horizontal Deformation with Respect to Horizontal Jack Load, Test Block No. 4

in which s is the total unit shearing strength, s_1 is the shearing strength under conditions of no load (pure shear strength), k is the factor of shearing-strength increase (assumed to be constant), and q is the intensity of load on the plane of shear.

Two interpretations of the data may be made. Interpretation *A* is based upon the use of only those tests in which the character of shale (as revealed by inspection after testing) is regarded as typical, and the neglect of those not typical, whether strong or weak; this interpretation excludes results obtained at Blocks Nos. 2 and 6. For Interpretation *B*, the results from all six test blocks are considered on the ground that prior to testing and subsequent examination of failure surfaces, it was believed that the character of shale at all blocks was similar, and that although the results obtained might not be consistent, the tests would be representative of the shale in its undisturbed form.

The shearing strength of the shale is appraised at a value intermediate between those of the respective interpretations. Using the data for the ob-

served failure loads, Equation (5) may be written (in units of tons per square foot):

$$s = 0.9 + 0.43\, q \quad \ldots\ldots (6)$$

Fig. 13(*a*) shows the position of the straight line defined by Equation (6), together with the plotted points. The expression for the shearing strength based upon the "estimated failure under sustained load" data is:

$$s = 0.8 + 0.35\, q \quad \ldots\ldots (7)$$

Fig. 13(*b*) shows the position of this straight line together with the plotted points from which Equation (7) was determined.

The sliding tests made after failure of the test blocks are the basis of the points plotted in Fig. 13(*c*). For the first four test blocks, sliding tests were made for two different intensities of vertical load. Assuming that s_1 in Equation (5) equals zero, a mean value of the coefficient of friction may be obtained for each block by constructing a straight line through the origin and through the average of the points obtained from the sliding tests for that block. Excluding the results at Block No. 2, an average value of the coefficient of friction of shale sliding on shale, parallel to the bedding planes, is 0.53, as shown by the slope of the straight line of Fig. 13(*c*).

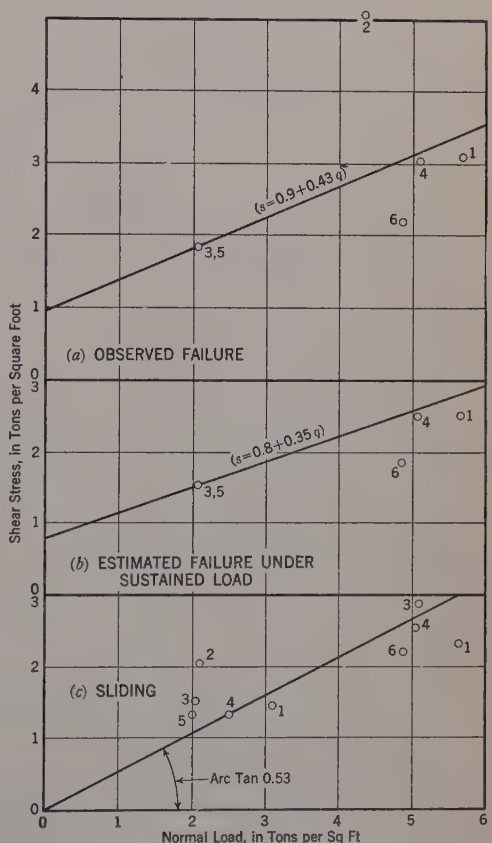


FIG. 13

The coefficient of friction so determined is probably too high; extension of the straight lines of Figs. 13(*c*) and 13(*a*) will indicate that, under higher normal loads, the resistance to sliding is greater than the total shearing strength. During the process of sliding, shearing off of projections on the failure surface continued; thus the construction of the straight line of Fig. 13(*c*) through the origin is probably not valid in this case.

The successful application of the Possum Kingdom test methods to the conditions existing for the Watts Bar tests affords additional proof of the value of Mr. Niederhoff's paper.

August E. Niederhoff,[7] Assoc. M. Am. Soc. C. E. (by letter).[7a]—One cannot but envy the delightfully simple routine of yesteryear's foundation engineer. His only field test for suitable rock was to hit it with a hammer and if the hammer rang, the rock was considered acceptable. Since that time loads have become heavier and more complex as superstructures have become larger. Engineers began to worry a little bit and took small samples of rock from the foundation to the laboratory where they could experiment on the "hand-picked" specimens. As long as the rock was hard and durable these laboratory tests were reassuring. No qualms of fear were felt that this quick analysis, wherein the specimen failed all at once like the poetic "one-hoss shay," did not give dependable results. Today, not only are the loads heavy but the foundation material is usually poor. There is also a dawning realization that there must be something to this fourth dimension concept sometimes called "time-effect" or "fatigue of materials." Also there is the growing suspicion that all foundations are not perfectly elastic media able to withstand the ravages of exposure; so the engineer again turns to field tests. The conduct of these field tests and the interpretation placed upon the results by the writer have elicited the views of several engineers. At least two cases in widely separated parts of the United States are known in which the field testing procedure, as described in this paper, has subsequently been used in whole or in part, yielding excellent results. The record is completed by the interesting and readable discussions of this paper that have added much to the store of knowledge on foundations.

Mr. Hatch contributes the observation that agreement between several laboratory results is reasonably good if they are properly evaluated. Using shear values derived from Equation (4) he finds the variation of consistency to be about 11 per cent. This is certainly within acceptable limits, but in the same paragraph he points out that laboratory shear values parallel to the bedding plane exceed those normal to the bedding plane. The expectation is naturally for results in the opposite direction and forms the basis for the writer's contention that laboratory tests should be taken with "a grain of salt." Too often they include numerous errors of sampling and duplication of natural conditions.

It is claimed that by experience Mr. Hatch has found that the coefficient of friction decreases with an increase in pressure. In Fig. 10 he has plotted shearing force against normal pressure, and the curved lines for both laboratory and field tests prove his point. The curves, of course, do not pass through the origin because there was evidence of definite adhesion between the concrete blocks and the shale. Another point that could be emphasized in Fig. 10 is the similarity of shape in the curves of the field and laboratory results. This unsuspected conformity between the two types of data is reassuring.

From a practical point of view it is debatable whether determining the angle of internal friction for a system composed of different materials such as concrete and shale, or for shale alone, is less important than knowing the actual shear value of the material. The writer believes both are important but that the

[7] Civ. Engr., U. S. Army Engrs., U. S. Engr. Office, Portland, Ore.

[7a] Received by the Secretary May 6, 1940.

answer in any particular case is necessarily influenced by the local conditions of contact between the concrete and the shale. For instance, a deep, narrow cut-off trench dug into the shale and filled with concrete to form a shear key presents one problem, whereas the same foundation using a pattern of small grooves cut into the shale to increase resistance to sliding is something quite different. Apparently the relative importance of shear values *versus* internal friction values must be decided on the merits of a particular case. In any event, determining the ratio of horizontal to vertical loads furnishes designers with a recognizable value. It is the kind of thing that gives them a comfortable feeling if they are sure their assumption is on the safe side.

For the foundation under consideration in the paper, the field data, and to a lesser extent the laboratory results, were enough to serve for the conclusions at the end of the section headed "Sliding of Concrete Blocks on a Shale Foundation." Criticism as to basing opinions upon a few tests can only be answered by the statement that even three field tests are preferable to having no results whatever. Limitations of money, time, and opportunity still harass the foundation engineer as well as other research professions.

The writer is pleased to note that Mr. Feld is of the same opinion regarding the dangers of extrapolating small laboratory tests to fit an entire foundation. The only time that this is excusable is when the failure values of small unconfined specimens are many times the actual loads to be placed on the foundation. Even then an engineer should run through preliminary calculations that tend to show that such a case actually exists. Table 3 demonstrates this procedure very nicely in addition to being an explanation of inconsistencies in the settlement curves in Fig. 8. Since writing the paper the author has noted that the presence of overburden loads in the proximity of the largest test plate may also have been responsible for erratic settlement. All of the test plates were placed in the bottom of a trench but the distance from the edge of the largest plate to the toe of the cut was a smaller proportion of plate diameter than was the case of the other two test plates. If the tests were to be done again the writer would see that no other loads were closer to the edges of the tested area than four diameters of the test plate. Mr. Feld is quite correct in his statement that tests and interpretations must be tempered by experience.

It is regrettable that discussion did not bring out actual measurements of settlement on Possum Kingdom Dam during construction. Data of this nature, if made available in the future, would be most conclusive as to the efficacy of the field and laboratory tests.

Mr. Rountree's discussion is clearly illustrated with results of tests conducted along similar lines and is one of the two aforementioned cases in which it is known that the test procedure has been used successfully. His observation that test blocks should not include the lateral support of abutting shale because of probability of erroneous interpretation of results is sound advice.

The fissile character of the foundation of Watts Bar Dam appears to make it a more difficult problem than that faced by the writer. The Tennessee Valley Authority has been known to adopt the best engineering methods in solving difficult design and construction problems and their decision to use field tests in this case illustrates the alertness of their engineering personnel. The

writer was fortunate in having the opportunity of discussing these experiments with their engineers at Knoxville, Tenn., prior to the actual running of the tests. Agreement was reached that by no other means could a more lucid picture be given of the results of biaxial loading of a laminated shale foundation. The use of six test blocks and the several observations of various phases during and after the experiments guaranteed the maximum amount of information from these tests.

Equation (5) is the generalization of Equation (3) of the paper; and, as pointed out in one of the other discussions, the relation between shear and normal pressure is not a straight line over the entire range. In Fig. 13(a), it is possible to draw a smooth curve through all points except 6, indicating a somewhat smaller value of shearing strength under conditions of no load than that obtained by Mr. Rountree's analysis. This smooth but irregular curve does not pass through the origin, confirming the fact that shear occurred in the shale and not principally along the contact plane of the concrete and shale.

Acknowledgments.—Tests described in the paper were made as a part of the investigation and design of Possum Kingdom Dam by the U. S. Army Engineers. After the completion of investigation and detailed plans the dam was built under the direction of a State agency employing a firm of private engineers. For the Army Engineers, Captain H. A. Montgomery was District Engineer in charge. Acting Principal Engineer, W. L. Kuehnle, Assoc. M. Am. Soc. C. E., exercised supervision over the conduct of the tests and offered valuable suggestions and observations. The part of the paper under "Geology" was prepared from the report of Warren J. Mead,[8] Affiliate Am. Soc. C. E., submitted to the district engineer.

The writer is pleased to acknowledge the excellent assistance of Logan Woolley, Jun. Am. Soc. C. E., and Mr. Edwin Jensen in the compilation and recording of data. The preparation of this paper and the interpretation of test results were among the duties of the writer. Messrs. Jensen and Floyd Johnson, formerly of the U. S. Army Engineer Office at Mineral Wells, Tex., directed the field work.

[8] "Geological Conditions at Possum Kingdom Dam Site," by Warren J. Mead, dated May 14, 1936, pp. 2, 3, and 4 (not published).

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FLOOD-PROTECTION DATA

PROGRESS REPORT OF THE COMMITTEE

Discussion

BY WALDO E. SMITH, M. AM. SOC. C. E.

WALDO E. SMITH,[7] M. AM. SOC. C. E. (by letter).[7a]—The Committee on Flood-Protection Data has submitted an interesting progress report summarizing the major developments in this field within the year 1939. The writer desires to mention a few points directly connected with the headings of the Report, and others concerning flood-control operation not mentioned.

Among the "Floods in Close Succession" might have been mentioned the period of the catastrophic flood of the Ohio Valley of January, 1937. Although it is true that on most of the Ohio River proper this was a single flood rising, sometimes gradually and sometimes rapidly, to the ultimate crest which followed "Black Sunday," January 24, on most of the tributaries this period bracketed a series of moderate to very high flood crests a few days apart. The general Ohio River flood so far over-shadowed these tributary floods, however, that the succession of floods has usually been overlooked.

The situation on the Muskingum Basin in southeastern Ohio may be taken as representative of tributary behavior. The period from January 13 to 24 included four distinct periods of rainfall, and on the Muskingum River at Zanesville, Ohio, four flood crests were produced, each successively greater than the preceding. On tributaries to the Muskingum River, there were likewise four distinct crests and, although the last was generally the greatest, they were not in most cases in the ascendent order mentioned for Zanesville.

On the matter of "Statistical Methods," the analysis based on past occurrences involves the principle that a flood of a certain great magnitude having occurred on a given basin, the probability of a future flood attaining this magnitude is apparently enhanced. It would seem that the opposite might be true. To illustrate the point, assume two basins lying side by side, identical in size, shape, climatological region, and every other detail. A very heavy rain

NOTE.—This Progress Report of the Committee on Flood-Protection Data was published in April, 1940, *Proceedings*. This discussion is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the Report.

[7] New Philadelphia, Ohio.

[7a] Received by the Secretary May 25, 1940.

chances to fall over one but touches only the fringes of the other. On the first, it produces the maximum flood of record, far in excess of any other flood; but on the second area, it causes only a minor rise, and the maximum flood of record remains one of much smaller magnitude. In drawing a frequency curve for each of the two basins on the basis of, say, their 100 years of record, the first area, apparently, would be subject to a much more severe flood hazard than the second; yet the two basins should have the same expectation of floods of this great magnitude. It would appear that, sooner or later, the second basin should expect a reversal of the condition bringing the record-breaking flood to the first basin so that the proper average will be attained ultimately. The second area, then, should have a somewhat greater expectation of future floods, rather than less as is indicated by the statistical method.

That the fallacies of the statistical method have been recognized is obvious in the work on the transposition of storms on which much work has been done by the Hydro-Meteorological Section of the Weather Bureau's River and Flood Division as mentioned in the report under "Hydro-Meteorological Studies," and by the Corps of Engineers, United States Army.

The Committee mentioned zoning regulations briefly. Parallel with these are the flood evacuation and relief plans that have been adopted by communities in the Ohio Basin and elsewhere. These plans are to be commended as a step in the right direction, but caution should be urged, especially during a series of flood-free years, to see that these plans are kept up to date; else they may prove to be worse than useless when need for them arises.

Under "Recent Publications" should be mentioned the series on "Hydrologic Studies" now being issued by the Soil Conservation Service of the Department of Agriculture, relating to the data gathered and investigations undertaken on their experimental watersheds over periods of five or six years. Worthy of note is the report of the Special Advisory Committee on Hydrologic Data of the National Resources Planning Board, titled "Deficiencies in Hydrologic Research," issued since the filing of the Progress Report under discussion. Also worthy of note and of recent release is the study by Robert E. Horton, M. Am. Soc. C. E., and Richard VanVliet on "Flood Volumes" issued by the Interstate Commission on the Delaware River Basin.

The two series of monthly publications issued by the Muskingum Climatic Research Center of the Soil Conservation Service should be mentioned. Both series begin with January, 1938. One is titled, "Precipitation on the Muskingum River Watershed, Ohio, by 30 Minute Periods," and the other, "Hourly Precipitation on the Upper Ohio and Susquehanna Drainage Basins." Unfortunately, the regular publication of these valuable records is hampered by lack of funds. The project under which these publications are being issued is worthy of considerable note by those interested in flood-control and water-supply problems. Because of its importance, and the infrequent reference to it in engineering literature, the following seems to be in order.

In the late winter and early spring of 1937, the Climatic and Physiographic Division of the Soil Conservation Service in cooperation with the Muskingum Watershed Conservancy District sponsored a WPA project to study rainfall and other meteorological occurrences on the Muskingum Watershed, and their

relation to stream flow and the physiographic features of the basin. This involved the development by the Soil Conservation Service of inexpensive recording rain gages, anemometers, and hygrothermographs. Five hundred recording rainfall stations were established and are now in operation on the 8,000-sq-mile watershed of the Muskingum River, giving an average spacing of about four miles; 250 hygrothermographs and between 150 and 200 anemometers record other occurrences. Thirty stream flow stations previously established on the basin by the U. S. Geological Survey, the Conservancy District, and other agencies provide means of checking runoff. Based on these records, series of precipitation, temperature, humidity, and wind velocity and direction maps have been and are being prepared in the project offices in New Philadelphia and Akron, Ohio. Of these, the precipitation maps had the widest demand, and publication of limited numbers was undertaken.

Utilizing the inexpensive recording rain gage designed for use on the Muskingum project, the Weather Bureau, in cooperation with agencies of the State of Pennsylvania, shortly established a more widely spaced network of recording rainfall stations covering the Upper Ohio and Susquehanna watersheds. By cooperating with these agencies, the narrow belt between the western boundary of Pennsylvania and the eastern boundary of the Muskingum Basin was closed in with a few stations. A few of the Muskingum recording stations, giving the approximate spacing of the network in Pennsylvania, were selected. On the basis of these data, the second series of publications was undertaken. The data collected, both published and unpublished, will give hydraulic engineers, hydrologists, climatologists, meteorologists, agronomists and others a mass of basic material that will mount in importance with the passage of years. Nearly three years of operation of the project have now (1940) been completed. It is to be hoped that funds for continued publication will become available to make these important records generally accessible.

While the current report makes no mention of the operation of flood-control works, this subject having been discussed briefly in the 1938 report,[8] the writer desires to call attention to and endorse those comments. Furthermore, it should be added that operation planning and control should involve an integrated organization embracing an entire basin, such as the Ohio Valley, or perhaps the entire Mississippi Valley. This organization should not be too much concerned with works under construction or with navigation works, but should try to make the best possible use of completed flood-control works. The recommendations of such an organization concerning flood-control needs in the way of projected works for more adequate protection, however, should merit serious consideration.

With further regard to the operation aspect, which has been a little-discussed subject to date, the following comments seem to be pertinent to the work of the Committee.

The development of flood forecast techniques for headwaters and downstream reaches alike has continued its forward progress in the offices of the various agencies concerned. This is important inasmuch as successful forecasting is a prelude to successful flood-control operation, and is the essence of the successful use of competent flood evacuation plans.

[8] *Proceedings*, Am. Soc. C. E., January, 1939, p. 98.

During the past three winters, the writer has prepared weekly snow cover maps for the Muskingum Basin. On that Basin, snow is usually not of great importance except as it melts with the occurrence of rainfall. An account was kept for each reporting station, setting down increments due to precipitation, and deductions due to thawing and runoff. Both water depth and snow depth were tabulated to insure that results did not become incongruous. The values were plotted on a base map each week and isohyetal lines were drawn, just as for rainfall occurrences. With the fall of rain and the melting of the frozen cover, proper amounts can be added to the rainfall to determine the total water on a watershed. During the flood of early March, 1940, on the Muskingum Basin, the map then current proved its validity and worth in planning the operation of the Muskingum flood-control works.

Still another effort to keep check on current conditions included weekly reports on the frost in the ground. The infiltration rate is probably close to zero as long as frost remains in the ground, but is likely to increase sharply with the thawing of the frost, thus affecting considerably the quantity of water available to produce flood conditions.

Many factors that seem small in connection with the design of structures assume major importance under operating conditions. Downspouts discharging their flow on to gate guides may build up ice, preventing the operation of gates. The proper design of ventilated galleries in masonry dams to eliminate the excess moisture that comes in on warm, humid days, at points at which it can be properly cared for, and the placing of mechanical and electrical equipment to be out of the zone of excess moisture should be given consideration. Gage wells should be constructed to permit ready access and flushing. Engineers who are working actively on the design of proposed structures should spend considerable time investigating the small problems of completed structures, and should strive to eliminate from later works as many of these embarrassing and aggravating difficulties as possible. The remedy is usually simple before construction starts if the designer is well versed in the problems involved.

It would seem that the adoption of the radiosonde for general use by the Weather Bureau for upper atmospheric observations presages the ultimate successful forecasting of rainfall amounts for general storms. Inasmuch as the starting point for flood forecasts, flood-control operation plans, and other activities connected with protection from damage by flood, can then be the atmosphere instead of measured rainfall on the earth's surface, this step is of utmost significance in the matter of flood-protection data.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

RELATION OF THE STATISTICAL THEORY OF TURBULENCE TO HYDRAULICS

Discussion

By A. A. Kalinske, Assoc. M. Am. Soc. C. E.

A. A. Kalinske,[63] Assoc. M. Am. Soc. C. E. (by letter).[63a]—The writer appreciates the many encouraging comments made by the various discussers, and especially the suggestions and advice submitted by experienced hydraulic engineers. These suggestions will serve as guides for further investigations in this field. Professor Rouse, who has given much thought to various theoretical and experimental phases of the turbulence problem, summarizes in a concise manner the items in the paper which relate to hydraulics problems. His effective analysis of the main features of the paper should prove helpful in dispelling fears that prospective readers may have regarding the presence of the more or less complicated mathematical formulations.

Verbal and privately written comments, other than the published discussions, seemed to indicate to the writer that many hydraulic engineers had the idea that the various theories and analyses presented in this paper had for their purpose the displacement of past ideas on water flow and the demonstration that various practical rules now in common use were all wrong. Nothing could be further from the truth. A study of the inner mechanics of fluid flow and attempts to understand the phenomena of turbulence have for their purpose the providing of basic, generalized information. It is hoped that such fundamental information will permit the bringing together of many apparently different hydraulic problems and will show that there is much that is common among them. Of course, some of the things that will be learned about turbulence will perhaps have no direct practical application. Much of such information will merely provide a better physical picture of the mechanics of turbulent flow. However, there are hydraulic problems for which an increased knowledge of turbulence is vital to their ultimate, complete solution. The recognition of

Note.—This paper by A. A. Kalinske, Esq., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Hunter Rouse, Assoc. M. Am. Soc. C. E.; February, 1940, by Messrs. Martin A. Mason, and J. C. Stevens; March, 1940, by Boris A. Bakhmeteff, M. Am. Soc. C. E.; April, 1940, by Messrs. Clyde W. Hubbard, John S. McNown, and Samuel Shulits; and May, 1940, by Messrs. Paul Nemenyi, and Bennie N. Netzer.

[63] Asst. Prof., Univ. of Iowa, Iowa Inst. for Hydraulic Research, Iowa City, Iowa.

[63a] Received by the Secretary May 7, 1940.

the practical importance of the various conceptions regarding turbulence that were presented is evident in the remarks of the discussers.

Mr. Stevens states that: "In uniform flow the boundary contacts are a source of continuing turbulence, a balance having been reached between that and heat dissipation." This is a significant and fundamental fact regarding the origin and dissipation of turbulence. Professor Bakhmeteff submits some very pertinent remarks relating to this phenomenon. There is no question but that the boundary regions in a straight conduit are the major source of the turbulence. The writer ventures the opinion that the physical law controlling the creation and shedding of the eddies from the viscous boundary layer in smooth conduits, and from the roughness projections in rough conduits, may some day be formulated. Professor Bakhmeteff raises the question relating to whether the intensity of turbulence is generally distributed across a conduit as shown by the writer's data. Repeated observations in smooth circular conduits by the writer and others have indicated definitely that the longitudinal component of the turbulent velocity fluctuation tends to reach a maximum in the region between the wall and about two thirds the distance from the center.

The writer appreciates the importance of having the proper fundamental concepts regarding the type of turbulence one is talking about, as was shown by Mr. Mason. True turbulence is probably best defined as being motion in which there is no periodicity in the fluctuation in magnitude of the three velocity components, v, u, and w. From a practical standpoint turbulence is nonisotropic if there is a mean velocity gradient normal to the direction of flow. Mr. Mason says that the present statistical theory of turbulence does not offer an immediate possibility of application to hydraulics problems since it is concerned chiefly with isotropic turbulence. This is not entirely true. One of the most important practical hydraulic problems depending for its complete solution on a better knowledge of turbulence is that of sediment transportation. Application of the concepts arising from the statistical theory has enabled the writer to study the mechanics of the placement of sediment into suspension in the bed region of rivers. Certainly the turbulence in this region is far from being isotropic. From the experimental standpoint, application of Professor Taylor's statistical theory of turbulent diffusion[41] has permitted the direct measure of the coefficient of diffusion in an open channel (see Fig. 18). A study of this coefficient is all-important in problems involving the transportation of suspended material. In general, so far as hydraulics is concerned, the statistical theory of turbulence has not provided and probably will not provide for a long time a direct and exact solution to any practical problems. However, it has provided and will provide tools with which hydraulic engineers will be able to get a better insight into the physical phenomena, and also will provide a basic framework from which orderly research can proceed.

Mr. Mason notes various other photographic techniques that have been used and suggested for studying turbulence. The size of the suspended immiscible liquid particles used by the writer averaged about 1 to 2 mm. It appeared that particles of this size were small enough for studying the diffusion

[41] "Statistical Theory of Turbulence," by G. I. Taylor, *Proceedings*, Royal Soc. of London, Vol. 151A, 1935.

and intensity characteristics of the turbulence in the main body of flow in pipes or channels. Studies near the boundary region require the use of much smaller particles, and a magnifying lens system is necessary for taking pictures in this region.

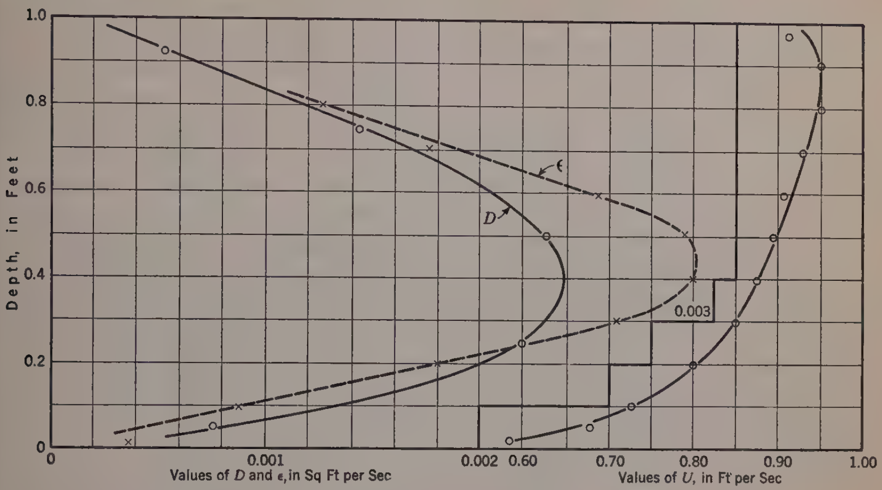


FIG. 18.—VARIATION OF D, ϵ, AND $\overline{U}$, IN THE CENTER OF A SMOOTH CHANNEL

Mr. McNown mentions that the turbulence must be considered in the study of sediment transportation as bed load. This is also pointed out by Mr. Nemenyi in his discussion. There can be no doubt that this is true, and fundamental studies of bed-load movement will have to give consideration to parameters which describe the turbulence quantitatively. Probably one of the most urgently needed fundamental experimental investigations in regard to sediment transport is a study of the zone near the bed where bed material is picked up and transported in suspension. Increased knowledge regarding this transitional zone in the bed region will be the key to the ultimate solution of the problems of prediction of the absolute concentration of suspended sediment. Professor Rouse has stated that:[64] "* * * analysis of the sediment problem as a whole will first become possible when bed-load and suspended load can be expressed as functions of the same flow parameters." In other words, the flow characteristics that control the movement of material along the bed also control the development of the turbulence that is responsible for the placement of material in suspension from the bed region.

The calculation of the length term λ, which is defined by Equation (30) in Mr. McNown's discussion, was made by substituting in this equation values of R_x for small values of x. This is in accordance with the procedure followed by Professor Taylor.[65] A knowledge of the functional relationship between x and R_x is not necessary in order to calculate λ to a degree of accuracy commensurate with the accuracy of the experimental data.

[64] "An Analysis of Sediment Transportation in the Light of Fluid Turbulence," by Hunter Rouse, U. S. Dept. of Agriculture, Soil Conservation Service, *Technical Publication No. 25*, July, 1939.

[65] "Statistical Theory of Turbulence," by G. I. Taylor, *Proceedings*, Royal Soc. of London, Vol. 151A, 1935, p. 456.

In the paper the writer makes a brief statement regarding the effect of turbulence on the measurement of static pressure in turbulent flow and the measurement of mean velocity by means of a pitot tube. From some measurements made by himself and various studies by others he concluded that for the ordinary turbulence present in pipe or channel flow the error in the "reading" of a static pressure tube or pitot tube due to the turbulence should not be greater than about 1%. Professor Hubbard, in his discussion, disagrees with this view. First of all it should be emphasized that the writer was talking about the error in the "reading" of a static-pressure tube or a pitot tube and not the error that may obtain in the use of these instruments in measuring the discharge in a conduit. There are many other items in addition to turbulence which affect the accuracy of discharge measurements made with pitot tubes.

The writer cannot agree that Mr. Hubbard[14] and Mr. Cole[37] in the paper mentioned have entirely succeeded in isolating the effect of the turbulence on the pitot-tube coefficients as obtained from over-all discharge measurements. The effect of the turbulence may have been obscured by various other items.

Probably the most thorough experimental investigation of the effect of turbulence on static-pressure tube readings was made by A. Fage.[66] Using the data obtained in this investigation, he made an analysis of the probable effect of turbulence on the calculation of pipe discharge from pitot-tube measurements.[67] The writer will not attempt to review completely the analysis made by Mr. Fage; however, a few conclusions should be mentioned in order to clarify some of Professor Hubbard's remarks.

According to S. Goldstein,[68] the reading of a total head tube in turbulent flow should be

$$H = \bar{h} + \frac{\overline{U^2}}{2\,g} + \frac{\overline{q^2}}{2\,g} \qquad (34)$$

in which $\overline{q^2} = \overline{u^2} + \overline{v^2} + \overline{w^2}$, the sum of the mean squares of the turbulent velocity components along the three coordinate axes; $\bar{h}$ is the local mean static pressure head; and U the local mean velocity. Mr. Fage points out that according to the vorticity transport theory the value of $\left(\bar{h} + \frac{\overline{q^2}}{2\,g}\right)$ is constant across the turbulent wake behind a body placed in a fluid stream. At the edge of the wake where the stream is undisturbed the static pressure is equal to $\left(h + \frac{\overline{q^2}}{2\,g}\right)$. This indicates that the true local static pressure varies across a turbulent wake. These conceptions are extended by Mr. Fage to the turbulence in a pipe, the main body of the turbulent stream corresponding to the turbulent wake. The reading of a static-pressure tube placed in a turbulent

[14] "Investigation of Errors of Pitot Tubes," by C. W. Hubbard, *Transactions*, A. S. M. E., Vol. 61, 1939, p. 477.

[37] *Loc. cit.*, p. 495.

[66] "On the Static Pressure in Turbulent Flow," by A. Fage, *Proceedings*, Royal Soc. of London, Vol. 155A, 1936, p. 576.

[67] "The Estimation of Pipe Delivery from Pitot-Tube Measurements," by A. Fage, *Engineering*, Vol. 145, June 3, 1938, p. 616.

[68] "Measurement of Total Head and Static Pressure in Turbulent Streams," by S. Goldstein, *Proceedings*, Royal Soc. of London, Vol. 155A, p. 570.

fluid, according to Mr. Fage's investigation, is equal to $\bar{h} + K(v^2 + w^2)$, in which K is an experimental constant which will depend on the design of the tube used. However, in any case the reading will tend to approach the value of the pressure at a smooth wall, h_w.

If the pitot tube reads a pressure equal to that given in Equation (34), and the other side of the differential gage is connected to a wall piezometer, Mr. Fage arrives at the following expression for the fractional overestimation of pipe delivery:

$$\int_0^1 \frac{\overline{q^2}}{\overline{U_m}^2}\left(\frac{\overline{U}_m}{\overline{U}}\right) d\left(\frac{y}{r}\right) + \int_0^1 \frac{(\bar{h} - h_w)}{\overline{U}_m^2/2\,g}\left(\frac{\overline{U}_m\, y}{\overline{U}\, r}\right) d\left(\frac{y}{r}\right) \ldots\ldots (35)$$

in which: $\overline{U}_m$ = mean velocity in cross section; y = distance from center; and r = pipe radius. Professor Hubbard's conclusion that if, as the writer states, a static pressure tube has a reading greater than the true static pressure by an amount proportional to the cross-velocity components, the coefficient of a pitot tube will be greater than unity, is erroneous. The effects of turbulence tend to make the coefficient of a pitot tube less than unity. By various transformations Mr. Fage reduces Equation (35) and gives as an approximate expression for the percentage of overestimation: $\frac{190\,\tau_0}{\rho\,\overline{U_m}^2}$, in which τ_0 is the wall shear and ρ is the fluid density. The maximum value of the expression occurs at the lower Reynold's numbers and is about 1.0%.

In addition to the effect of turbulence, Mr. Fage shows that an overestimation of discharge results from pitot-tube readings due to the fact:

> "* * * that the total head registered by a pitot tube in a region of total pressure gradient is not associated in general with the geometric center of the mouth of the tube, but with an effective center which is displaced from the geometric center towards the region of higher total head."

However, this correction should ordinarily not exceed 0.5% except when a large tube is used in a small pipe. Another item that may influence the reading of a static pressure tube is the deflection of the stream lines of the average flow by the presence of the tube in a region of high velocity gradient—that is, near conduit walls.

Everything considered, Mr. Fage's studies should not be considered as the final word in regard to the effect of turbulence on the readings of pitot tubes and static-pressure tubes; however, his analyses appear to be fundamentally sound and indicate the factors that must be studied in any experimental work on this particular problem.

In his discussion Mr. Shulits wonders whether it is a law of statistics that deviations from the mean of any large number of measurements will be distributed according to the normal error law. This is not true since, if (for instance) the fluctuations were periodic, the distribution of the deviations would certainly not be according to the error law. The fact that velocity fluctuations in fully developed turbulent flow are distributed according to the normal error law is a definite characteristic of turbulence, and not just a mere coincidence or the general behavior of any fluctuating quantity

Mr. Shulits states that the eddy viscosity, ϵ, which is proportional to the coefficient, D, as defined in Equation (6), is a time rate of change of area. Professor Taylor[6] definitely shows that the diffusion coefficient is proportional to the product of the root-mean-square of the transverse velocity and the distance a mass of fluid travels before the correlation coefficient between the velocity at the beginning and at the end of the distance is zero. To state that this coefficient is a time rate of change of area has no physical significance and is merely a "play" on the dimensions of this quantity. The kinematic viscosity, ν, of fluids has the same dimensions as ϵ or D—that is, $\frac{L^2}{T}$. A mere knowledge of the dimensions does not permit a physical interpretation of ν. However, analysis of the physical processes by which, for instance, a gas exhibits the property of viscosity, definitely shows that ν is proportional to the product of the mean velocity of the gas molecules and their mean free path. Professor Taylor's analysis gives a similar physical significance to the quantity ϵ or D.

Experimental data on the variation of D and ϵ in the center of a smooth open channel, 1 ft deep and 2.5 ft wide, are shown in Fig. 18. The mean velocity in a vertical section is 0.865 per ft sec. Values of D were obtained from data similar to those shown in Fig. 6 and ϵ was computed from the relation:

$$\tau = \frac{\rho \, \epsilon \, d\overline{U}}{dy} \qquad (36)$$

The computation of ϵ was not very accurate since some assumptions had to be made regarding the value of the shear. Nevertheless, the proportionality and perhaps even the equality between D and ϵ are evident.

The writer agrees with Mr. Shulits that the statistical theory of turbulence does not in any way obviate the necessity of observations and measurement; however, it does give direction to such research work.

Mr. Nemenyi makes a very worthy contribution in regard to the retardation of flow in conduits having various types of boundary obstacles. He remarks that energy dissipation studies will have to go beyond turbulence study and into the concepts of vorticity, boundary layers, and discontinuity surfaces. This is true, but it should be added that turbulence studies in general must also go back to these concepts. Since turbulence tends to originate at boundaries and surfaces of discontinuity, the laws controlling the creation of vortices will have to be studied if hydraulic engineers are to learn more about the physical processes in regard to the origin of turbulence. Mr. Nemenyi's remarks relating to the various types of surfaces of discontinuity that may be produced are very enlightening.

Present knowledge seems to point to the fact that ordinary turbulent flow in conduits is the result of the dispersion of eddies from the boundaries into the main body of the fluid stream. Their origin, whether from a surface of discontinuity at the viscous boundary layer in smooth conduits or from roughness projections in rough conduits, is probably governed by strict physical laws. The behavior of an individual eddy in the main fluid stream, of course, is unpredictable.

---

[6] "Statistical Theory of Turbulence," by G. I. Taylor, *Proceedings*, Royal Soc. of London, Vol. 151A, 1935, p. 421.

In order to retard the mean forward velocity of flow in a uniform conduit having a given hydraulic gradient and cross section of flow, the total rate of pressure or potential energy conversion must be decreased. This total rate of energy conversion per unit length of conduit is equal to $Q \gamma S$, in which Q is the discharge, γ the specific weight, and S the hydraulic slope. According to Equation (21), this rate of conversion for ordinary turbulent flow in a circular conduit can be represented by $2 \pi \int_0^r \rho \epsilon \left(\frac{d\overline{U}}{dy} \right)^2 y \, dy$. The relation between ϵ and $\frac{d\overline{U}}{dy}$ is expressed by Equation (36). The average shear will remain constant if the cross section of flow and S remain the same. Any change in ϵ should result in a proportionally equal and opposite change in $\frac{d\overline{U}}{dy}$. From the foregoing expression for pressure energy conversion (see Equation (21)) it is seen that an increase in ϵ will result in a decrease in Q, or in the mean velocity for a given cross section. In other words, to reduce the mean forward velocity it is necessary to secure intense mixing between the slowly moving water at the boundary and the water in the center portion. This can be done by an increase in the diffusion coefficient, ϵ.

Ordinary roughness projections, as pointed out by Mr. Nemenyi, are not effective for producing a large retardation of the forward velocity. Specially designed obstacles that forcibly throw the water at the boundaries into the main portion of the stream are much more effective in retarding the mean forward velocity of the stream. Though the flow in such special conduits is obviously not directly comparable to ordinary turbulent flow, nevertheless the general concept that intense transverse mixing must be obtained, if a large reduction in forward velocity is to be secured, is still applicable.

In his discussion of the application of the theories of turbulence to problems of sediment transport, Mr. Nemenyi states that it is paradoxical that various regular phenomena manifest themselves in turbulent streams in which the velocity fluctuations are supposedly irregular. Difficult as it may be to imagine how irregular fluctuations can create regular phenomena, nevertheless it does appear that turbulence may even have a direct influence in the production of such effects as for instance the formation of sand ripples on a river bottom. In a paper[69] published in 1939 M. A. Velikanov gives a mathematical analysis which seems to indicate that a plane sand bottom in a uniform stream is transformed into a rippled one by the direct action of the turbulence of the stream.

In regard to Mr. Netzer's comments on the measurement of fluctuating velocities in model studies it should be kept in mind that the photographic methods, as used by the writer, have many advantages even if they are time-consuming. Since no apparatus is introduced into the stream, there is definite assurance that the results obtained are not influenced by, or are peculiar to, the measuring apparatus. Furthermore, the investigators obtain a definite mental

[69] "Formation of Sand Ripples on the Stream Bottom," by M. A. Velikanov, *Question 3, Report 13*, published for meeting of International Geophysical Assoc., September, 1939, Washington, D. C.

picture of the turbulence, which is often as important as the actual quantitative measurements. The time required in analyzing the data obtained on motion-picture film can be shortened by utilizing the principles of random sampling theory. Nevertheless, any instrument, capable of measuring even approximately the turbulent velocity fluctuations directly, would be useful. A checking of the measurements obtained with an instrument such as that shown in Fig. 17, against data obtained by photographic studies, would indicate how the instrument responds to the velocity fluctuations in turbulent flow.

It should be mentioned that excellent data on velocity fluctuations in large streams can be obtained with current meters, especially with those of the smaller type. Although, to a certain extent, the data are qualitative, they nevertheless permit the study of the relative magnitude of the velocity fluctuations under different conditions. One of the problems in which a knowledge of the magnitude and frequency of the velocity fluctuations in natural streams is of importance is in suspended material sampling. Before suspended material can be determined accurately and economically, it is important to know how the concentration fluctuates; this fluctuation in sediment concentration is closely allied with the velocity fluctuations of a turbulent stream.

Acknowledgment.—The data for plotting Fig. 18 were taken from "Diffusion Characteristics of Turbulence in an Open Channel," by James M. Robertson, Jun. Am. Soc. C. E., a thesis presented to the University of Iowa in January, 1940, in partial fulfilment of the requirements for the degree of Master of Science.

Correction for *Transactions:* May, 1940, *Proceedings*, page 970, line 6, change "curves" to "figures"; and, on page 973, line 3 following Equation (32), change "12(*a*)" to "12(*b*)" and "12(*b*)" to "12(*c*)."